# Engineering News-Record

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# Of *WHAT* Capacity?

FROM time to time someone quotes to his own purpose the old wheeze about there being three grades of liars: liars, damned liars and statisticians. Without endorsing in the slightest degree so salty and sweeping an appraisal of the dealer in statistics, it is worth while to recall that his most honest and well-intentioned efforts sometimes are betrayed by the raw materials of his craft.

We live in a fast moving age. Materials, machinery, management and men all have a fashion of becoming obsolete with the march of progress. From this tendency even statistics are not exempt, especially those that may embody standards and relationships. The basic factors behind such statistics may change so substantially that even Einstein himself might be hard put to keep them accurate.

Something of this is coming to be recognized in the case of certain statistics that have to do with industrial activity. One of these is the ratio of current operations to capacity. Frequently it is stated and taken for granted that this, that or the other industry is operating at 50 per cent or 60 per cent of capacity. Of *what* capacity? Physical capacity? Economic capacity? When and by what standards was capacity measured?

DURING recent years, obsolescence has taken heavy toll of America's producing plant. It is still taking it and will continue to take it so long as scientific and technical progress persist as major factors in our industrial activity. This applies to almost every field of industry and industrial service. So far as concerns economical production at modern standards of cost and quality, many existing plants today are out of the running. When we include them in an estimate of the industry's current capacity, we are but deceiving ourselves. Capital may have been invested in them; the plant itself may still be in existence. Physically the capacity may be there; economically it may be as non-existent as though it had been dumped into the sea.

Even in the individual plant certain units may not be on a par with the production standards of other units; the economic capacity of the entire plant, again bearing in mind costs and quality, may be something much less than the sum of the figures of the equipment name-plates Possibly the plant layout and the design of its buildings may discount appreciably the nominal capacity of its installed equipment. It is true that in the individual plant a centralized management probably is alert to remedy such conditions and to keep the plant as a whole in economic balance; yet, despite all this, the "bottleneck" is not unknown even in the single plant. To avoid just that is one of the major problems of management.

BUT for any industry as a whole, with its many independent plants subject to no general coordination, there can be little hope of maintaining an effective economic capacity on a par with a nominal physical capacity. Self-deception on this score is unfortunate. It is quite likely to induce a complacency with respect to existing conditions; it may lead to false conclusions as to the adequacy of existing facilities. If these are accepted as the basis for our industrial development they may well foster the survival of obsolete marginal plant, greatly to the disadvantage of both consumers and the more progressive producers.

Right now, as we enter upon a general industrial expansion, is a good time for statistical agencies, economic advisers, industrial associations and individual industrialists to undertake new inventories of industrial capacity so that we may be able to formulate our planning on the basis of a more accurate relationship between capacity and prospective demand and to proceed more confidently with the provision of modern facilities to meet that demand.

Willard Chevalier

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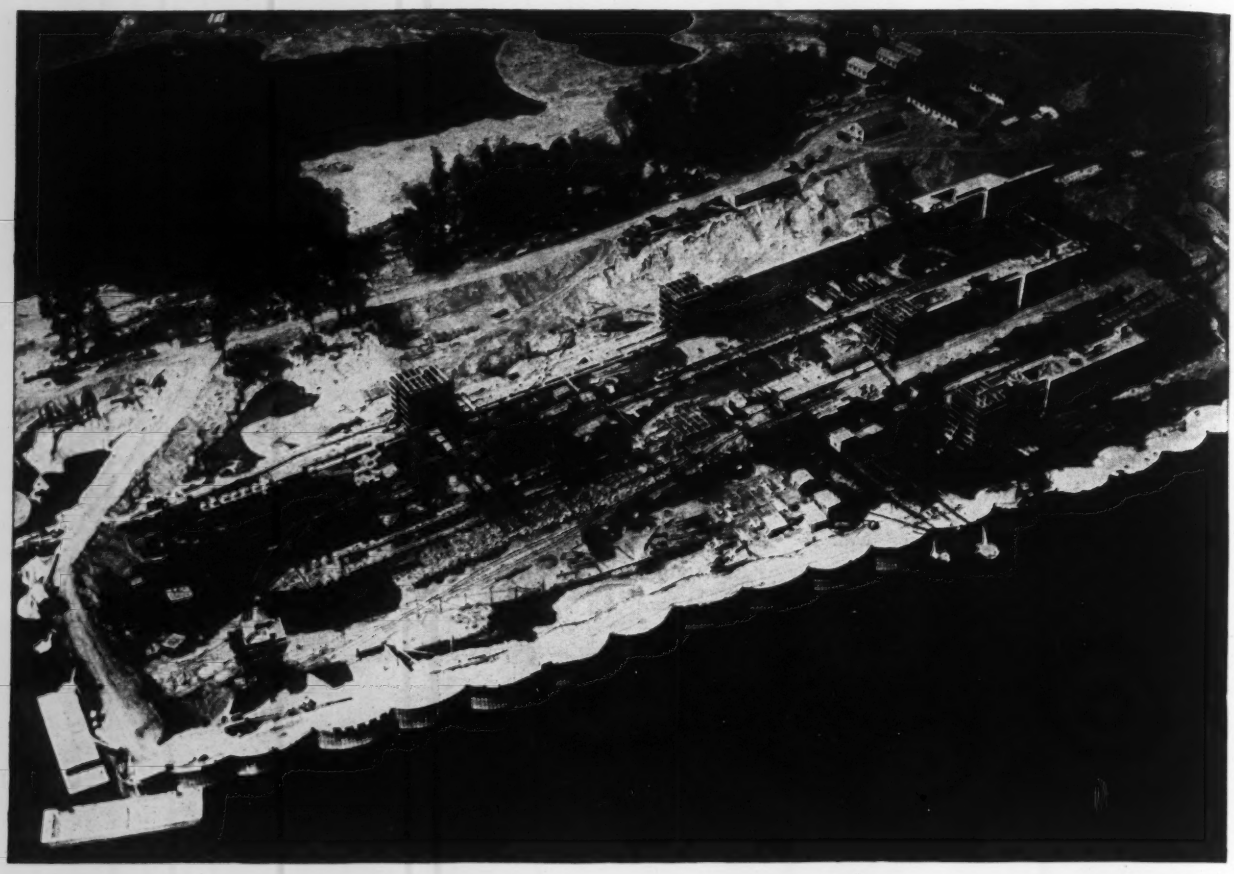

# Inland Piling at Lock 25 On The Mississippi

*Above:* Construction photo of Mississippi Lock No. 25 at Cap Au Gris, Missouri. 1751 tons of Inland Section I-23 was used, some of the piles being 65 feet long.

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The view clearly shows the neat tight job of Inland Steel Sheet Piling installed by the United Construction Co. United reports everything going well and work progressing splendidly.

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**Arteries of a Sewerage System**—Chicago is completing the last major link in its vast network of sewer tunnels with the driving of the huge Southwest Intercepter in preparation for reducing diversion of sewage dilution water from Lake Michigan. Driving procedure of the 12½-mile tunnel will be described in next week's issue.

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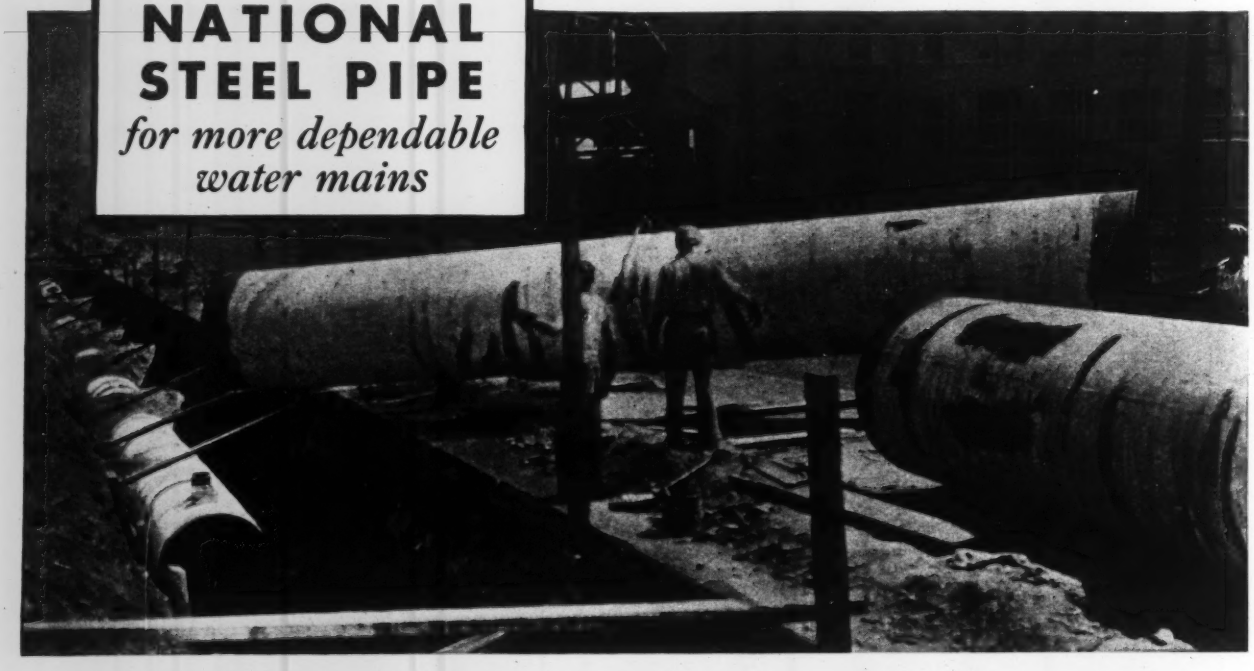

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November 12, 1936

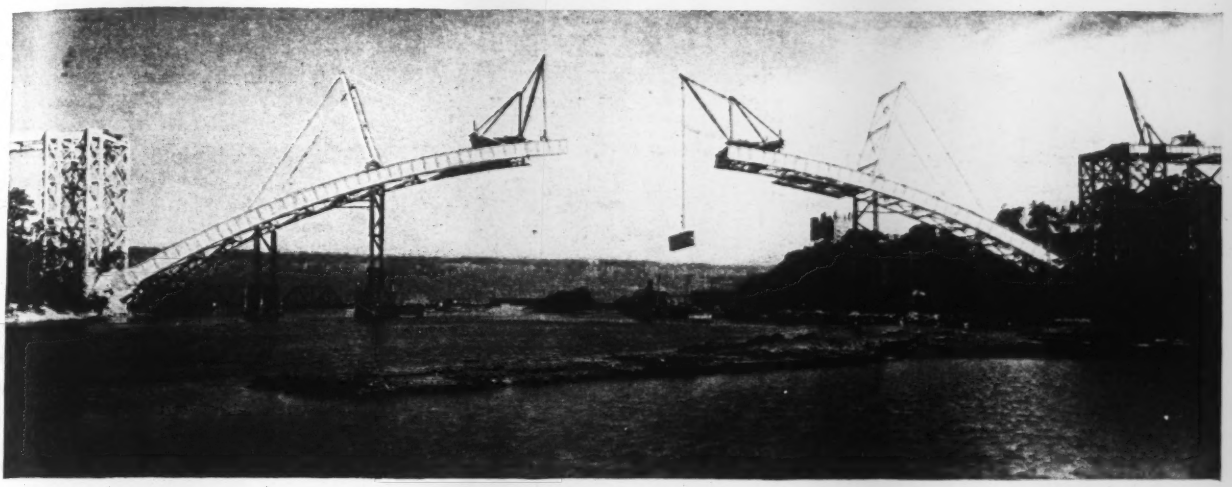


FIG. 1—RIB SECTION being lifted from barge by one of the erection travelers on the Henry Hudson Bridge. Toggles permitted center 420 ft. of arch to be erected without falsework.

# Special Problems Of Hingeless Arch Erection

**Steel arch of 800 ft. span for Henry Hudson Bridge in New York required unusual fabrication accuracy and careful crown jacking to induce proper stress conditions in the finished structure**

By W. K. Greene,
*Assistant Engineer, American Bridge Co., New York, N. Y.*

THE ERECTION of the 800-ft. central span of the Henry Hudson Bridge in New York, the longest plate girder arch as well as the longest fixed arch in the world, is notable in several respects: the hingeless arch construction imposed unusual requirements in the fabrication of the arch ribs, and necessitated accurate control of the stress conditions during the various stages of erection; the closure procedure involving the insertion of a crown piece gave rise to a series of jacking operations somewhat out of the ordinary; and, finally, maintenance of navigation in the Harlem River and of railroad traffic on the New York Central Railroad tracks which pass under the arch span on the north side of the river required such location of falsework bents that rather long cantilever arms resulted, necessitating the use of adjustable toggles over two of the falsework bents.

The Henry Hudson Bridge and its design were described in *Engineering News-Record*, Aug. 13, 1936, p. 232. In brief the arch span consists of two box girders 12 ft. 6 in. deep by 5 ft. 6 in. wide spaced 50 ft. centers. The deck which is supported on spandrel columns carries a 42-ft. roadway and a 4-ft. sidewalk. The main section and splice material of the arch ribs are silicon steel. All other material is carbon steel. The arch span was designed as fixed for full dead and live load. The effect of rib shortening was considered for live load only since the ribs were cambered for full dead load, and temperature stresses were figured on the basis of a normal temperature of 55 deg. F.

In order to realize the proper stress condition in the completed structure, it was therefore necessary to fabricate and erect a structure which would have the proper shape under full dead load at normal temperature. A common method (originally specified for this span) used to achieve certain stress conditions in a structure, is to have a closing member the length of which is based on observations made in the field after the structure is partially erected. Observations of this sort are always affected by temperature, wind and other uncertain factors and often cause hazardous delays in the field. The problem in this case was further complicated by the fact that in addition to axial load a considerable bending moment also had to be induced in the arch ribs at the time of closure.

After a careful study it was concluded that the desired stress condition would be more nearly realized by measuring the actual span length in the field after the main piers were completed and the grillages placed and by accurate fabrication of the arch ribs to a shape and length to fit this span length.

This method also permitted complete fabrication and assembly of all arch ribs in the shop and avoided considerable de-

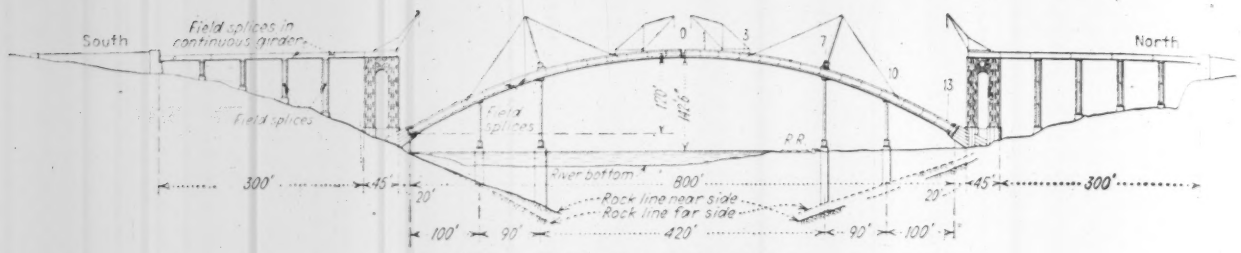



**FIG. 2—ERECTION DIAGRAM for 800-ft. hingeless arch and continuous steel girder approaches of Henry Hudson Bridge at New York.**

lay in the field. For reasons which will be outlined later in this article, it was necessary to have a "keystone" or closing piece about 2 ft. 6 in. long at the crown of the arch span. The fabrication of this closing piece was held up until the actual span length had been determined and measurements made on the assemblies of the arch ribs at the shop. The actual length of closing piece required was then determined and the piece was fabricated and assembled in the shop with the adjoining rib sections.

## Fabrication

The arch ribs were fabricated in panel lengths except the two end sections which were made shorter to reduce the weight. The lengths of the rib sections varied from 23½ to 32½ ft., and they weighed 38 to 48 tons each.

Both ends of all rib sections were milled for bearing, the splices being on radial lines. The milling of one end over an area of 12 ft. 9 in. by 5 ft. 6 in. was accomplished in one setting of the piece. Extreme care and accuracy were exercised in the milling as an error in angle at any one joint would cause an error many times larger in the curve of the assembled sections. The rib sections were set up for milling to calculated bevels and angles, and check measurements were taken on the sides and diagonals of the trapezoid formed by the extreme four corners of the finished piece.

Shop assembly of the rib sections in lengths not less than 200 ft. was required. Measurements were taken on each assembly to check the accuracy of fabrication, and the splices were then reamed. Sections were added to one end of an assembly and removed from the opposite end so that at least 200 ft. of rib was assembled at all times. The check measurements consisted of measuring the total overall length and the ordinate at each panel point. As a further check the angles formed by the milled ends and the chord of the assembly were checked for eight assemblies. The arch ribs as fabricated were very close to calculated dimensions. The total length of rib, 846 ft., overran less than 1 in., and in general the ordinates of the curve were within a total variation of ¼ in. This variation in length was taken care of in the closing piece.

In arriving at a general erection procedure, consideration was given to various falsework and cantilevering schemes and methods of closure. The procedure adopted is indicated in Fig. 2. The fixed arch construction was utilized to cantilever out from the main piers for the first stage of erection. Falsework bents were placed at panel points 7 and 10 north and south, and toggles were provided at panel points 7 to take the bending moment in excess of the capacity of the arch ribs and splices. The closure method chosen was closure at the crown of the arch by means of loads applied externally by jacks at that point and if required by manipulation of bents 7N and 7S also.

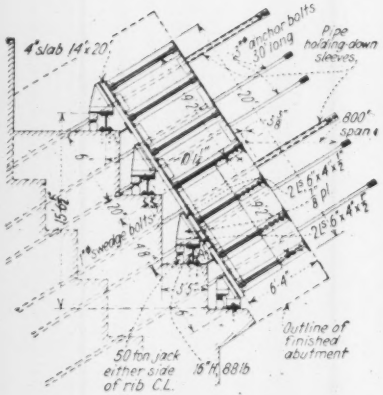

4" slab 14'x20'
Anchor bolts 30" long
Pipe holding-down sleeves
800' span
Outline of Finished abutment
50 ton jack either side of rib CL.

**FIG. 3—GRILLAGE ERECTION at each rib abutment required extreme accuracy which was aided by a jacking arrangement that permitted moving the heavy assembly in any direction.**

In order to keep the loads on the main piers, falsework bents and toggles within reasonable limits it was necessary to erect the arch ribs low and provide a closing piece to be inserted after the closure jacking had raised the ribs to final position and increased the opening at the center to the proper amount. The length of closing piece, 2 ft. 6 in., was determined on the basis of a 6-in. clearance before closure jacking, to take care of adverse temperature conditions and other variations.

The erection stresses for this procedure did not require any change in the design for the main piers or arch ribs, the only variation from the design being the addition of diaphragms at the bents and connections for the toggle. This was highly important as any change in the design would have required refiguring the permanent bridge for the new sections.

Referring to Fig. 2 it will be noted that the north one-third of the arch is over land and the remainder over the Harlem River. The grillages, skewbacks and arch ribs out to bent 10S were erected by a derrick boat. The corresponding members on the north side were erected by a locomotive crane. The remainder of the arch ribs were erected by two travelers, working from either side to the center of the span and riding directly on the cover plates of the inner girders of each arch rib.

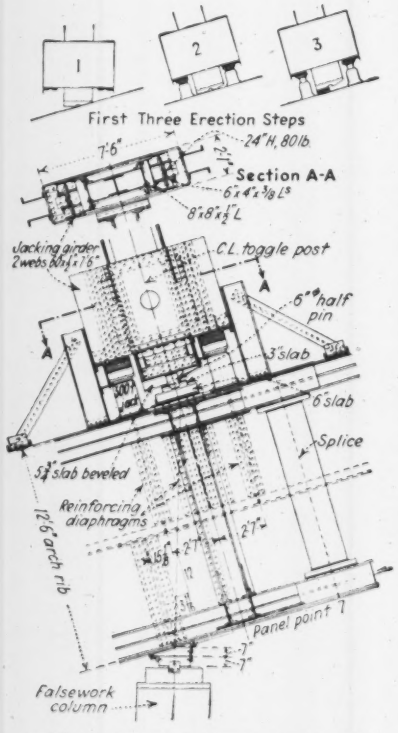



**FIG. 4—TOGGLE POST details with erection steps indicated.**

## Setting the grillages

The fixed arch construction required accurate setting of the grillages to elevation, angle and transverse and longitudinal position. The grillages which are 20 ft. by 14 ft. by 6 ft. 4 in. were too large for assembled shipment and had to be delivered to the site in two pieces.

These were set in approximate position and bolted together prior to final adjustment. The jacking arrangement (Fig. 3) was such that the grillages could be moved in any direction thus facilitating the adjustment to perfect position. After each grillage was set in final position, it was firmly secured against displacement by tightening four of the eighteen 3-in. diameter, 30 ft. long anchor bolts and welding steel angles between the grillage and the concrete in the main pier. The reinforcing bars were then put into place around the grillage and the concrete encasement placed. The actual span length was determined after all grillages were encased ready to receive the steelwork.

### Falsework bents

The falsework used for the bents was entirely of steel construction and adjustable to accommodate any ordinary span width or bent height. A jacking base with two hydraulic jacks was located at the bottom of each column. The grillages supporting the bents framed directly to steel H-beam piles with cast steel tips which were braced from the grillage to the river bottom by a falsework cage.

The cages for bents 7S and 10S were assembled on barges at a convenient location away from the bridge site. At the proper time these were towed to the site and placed in position on the river bottom by a derrick boat. The cage for bent 10S weighed 80 tons and the one for 7S 140 tons (Fig. 5). Falsework cages were also used on the north side, although the bents were on land, in order to facilitate driving the piles. The lengths of the piles varied from 60 to 135 ft.

Two spandrel columns borrowed from the bridge deck framing were utilized for the toggle bents, and a special jacking bracket was provided at the bottom of each column with a capacity equal to the maximum load. This jacking arrangement (Fig. 4) was used to bring the toggle into play and to adjust the arch ribs. The toggle itself was an eyebar chain, two bars 10x1-7/16-in. to each chain. The toggle bents were made short so that the eyebar chains could easily be connected and then tightened by jacking up on the bents.

### Setting the steel

The erection from the main piers out to the center of the arch span was ordinary cantilever erection procedure. The skewbacks were placed and bolted and doweled to the encased grillages. The eighteen 3-in. diameter anchor bolts in each pier were then tightened up with a special wrench 6 ft. long. The arch ribs were then cantilevered on the anchor bolts to panel point 11 and falsework bent 10 placed. Arch ribs 11-10 were next erected and supported on bent 10 and at panel point 11 of the cantilever

**FIG. 5—FALSEWORK CAGE** weighing 140 tons being lowered to the river bottom by a derrick boat. Steel H-piles driven through this cage supported the bents as shown at the left.

with the splice at 11 open at the top. Bent 10 was jacked up to close the splice at 11 and then jacked up $\frac{5}{8}$ in. higher to relieve the bending moment on the skewback.

The cantilevering continued to panel point 7, the location of the next falsework bent, after which this bent was jacked up $9\frac{5}{8}$ in., to remove all load from bent 10. Then, before erection proceeded, the toggles were erected with the posts vertical and the eyebar chains connected to the saddles (Fig. 6). Continuing erection to panel point 4, bent 7 was jacked down $5\frac{7}{8}$ in. to relieve the bending moment at the skewback. Arch ribs 4-3 were erected and the forward ends of the eyebar chains connected. The toggle bent was tipped back to be vertical with the rib axis, the rear ends of the eyebar chains connected and the bent jacked up 1 ft. $4\frac{1}{4}$ in. to tighten the eyebar chains. Erection continued to panel point 1, and bent 7 was again jacked down ($8\frac{3}{8}$ in.) to relieve the bending moment at the skewback.

At this time the alignment and elevation of the arch ribs, which had been watched closely during the erection, were checked, and slight changes made from the theoretical jacking down of bent 7 to obtain perfect alignment of the two halves of the arch and equal elevation for the ends of the four ribs. Arch ribs 1-0 were erected, the toggle bents jacked up or down as required to obtain perfect vertical alignment of the ribs, and the guides and jacking equipment placed at the crown.

### Crown jacks

Since the closure jacking involved creating an opening for the insertion of the closing piece, the jacks and blocking shims were placed outside of each

**FIG. 6—TOGGLE BENT** being erected as traveler moves out onto the cantilever arm. After two more rib sections were erected the toggle chains were connected and the posts tipped back normal to the rib centerline.

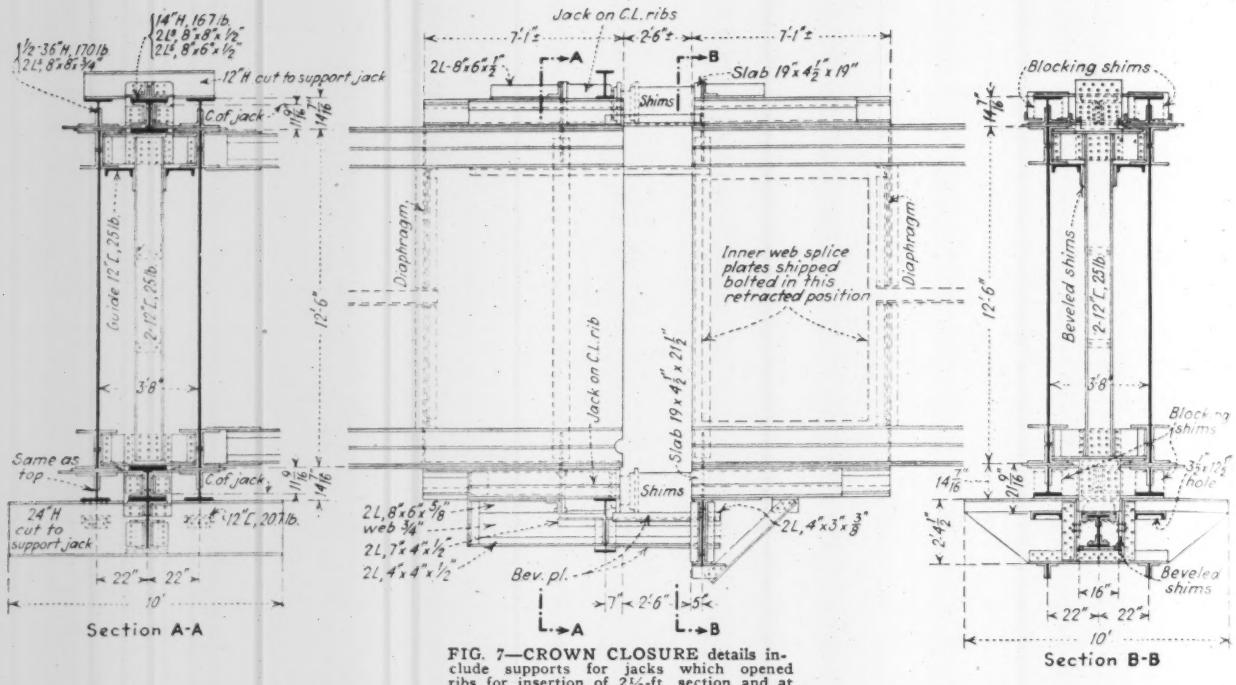



FIG. 7—CROWN CLOSURE details include supports for jacks which opened ribs for insertion of 2½-ft. section and at same time induced proper stress condition in ribs. Small pins in lower chord assisted in closing the center splice along its full length.

flange to avoid interference (Fig. 7). During the jacking procedure the crown of the arch rose about 4½ ft. causing an angular change in the relation between the two halves of the arch span. In order to maintain a truly axial load until after the erection of the closing piece, the upper and lower jacks were interconnected and operated from the same pump. It was desirable to keep the load on the jacks parallel to the plunger, and this was accomplished by means of a system of beveled shims which were changed from time to time as the angle between the two sides changed. Further to protect the jacks from eccentric loads, slabs with a bearing area 6 in. wide at the center of the jack were inserted in the shim piles. Similar arrangements were provided for the blocking shims, and wedges which could be driven in tightly permitted following the jacking very closely as the opening increased (Fig. 8).

The ends of the ribs were lined up accurately at the start of the closure jacking, and guides maintained that alignment both vertically and transversely. It was anticipated that the first contact on release of the jacks would be at the bottom of the arch rib. Small pins, therefore, were placed in the bottom flange to serve the double purpose of final alignment and to induce sufficient moment from eccentrically applied load to close the center splice over its full depth.

## Alternate closure plans

Temperature at the time of closure has a marked influence on the magnitude of jacking loads. On account of the uncertainty as to when the closure would be made, two methods of closure were worked out in complete detail; one to be used if the closure should be made in cool weather, and an alternate for warm weather. For the first closure scheme the jacks at the crown of the arch only were to be employed. The load on bents 7N and 7S would gradually be reduced as the opening at the center increased to 9 in., and from that point on to the full opening of 2 ft. 6 in. the crown jacks would carry the entire load, which at normal temperature of 55 deg. F. would be about 1200 tons for each rib.

FIG. 8—SHIMS AND WEDGES on top chord which held opening as jacks were reset.

For the alternate closure scheme the jacks at bents 7N and 7S would be used as well as those at the crown. The crown jacks would first be brought into play and loaded to about 1000 tons per rib. This would increase the opening at the center 7 in. and considerably reduce the load, on bents 7N and 7S. From this point on, the jack both at the crown and at the bents would be operated simultaneously keeping the crown jacks loaded at all times to 1000 tons per rib. The toggle load for both closure schemes would be reduced by the jacking procedure from a maximum at the start to about 60 tons for each eyebar chain at the end.

The closure of the arch span was accomplished on July 8, 1936, at which time temperatures up to 90 deg. F. obtained. The alternate closure scheme, therefore, was used, to avoid excessive loads on the crown jacks. Work was started at 5 A.M. so that the temperature during the jacking would be most advantageous. Power pumps operated by air were used and all jacks were equipped with pressure gauges. All jack movements were directed by telephone from a central post at the crown of the arch.

The crown jacks were jacked up to 1000 tons per rib (9000 lb. per sq. in. for each jack), and measurements taken on the increase in opening and angle of the ribs at the center and also on the load remaining on bents 7N and 7S. These bents were jacked up ½ in. and the crown jacks maintained at 9000 lb. per sq. in. Measurements were again taken on the opening to check on the elastic behavior of the structure. This was fol-

lowed by another ½-in. increment, after which the jacking on the bents proceeded by 1-in. increments up to 12 inches. Check measures were taken and recorded after each increment. The theoretical relation of movement for the jacks (1 in. on the bents to 1⅛ in. at the crown) was realized almost perfectly.

It was necessary to fleet the crown jacks three times to obtain the full opening of 2 ft. 6 in. plus ¾ in. clearance, to erect the closing piece. At these times the beveled shims were changed as required to compensate for the change in angle between the ribs as the arch was raised to final position.

The jacking proceeded smoothly and the load was placed on blocking shims with the opening ready to receive the closing pieces by 11 A.M. The closing pieces and web splice plates were placed and bolted and pinned on one side. The load was then picked up by the jacks at the crown and bents 7N and 7S were slacked off to bring the closing piece into bearing, thereby creating a fixed arch. The aligning devices functioned perfectly and at 2:30 P.M. the arch was closed. The other side of the center splice was bolted and pinned and the web splice plates between the flange angles placed. Finally bents 7N and 7S were jacked down transferring the load entirely to the main piers. The remaining steps in the closure were removal of the jacking brackets and placement of the permanent flange splice plates, cross-frame and floorbeam.

The arch rib splices were riveted as the erection progressed and a schedule was worked out whereby the riveting would be done under a condition of minimum moment.

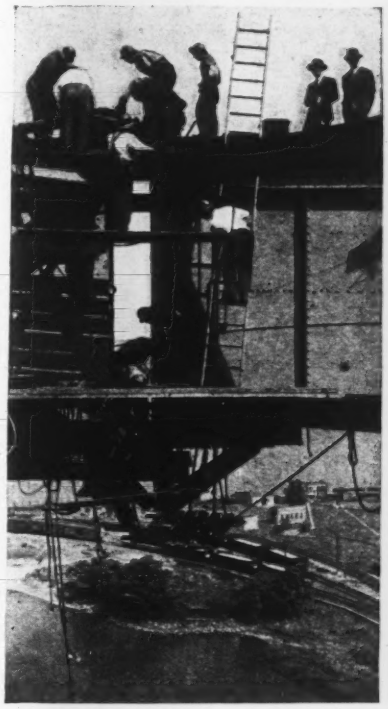


FIG. 9—OPENING AT CROWN complete, ready for closing section of rib

### Deck construction

The scheme for placing the arch deck and the concrete roadway was determined with a view to avoiding excess unbalanced loading on the arch ribs. The south arch traveler erected five panels of arch deck, starting from the center, and removed the south toggle while the north traveler removed the north toggle. A viaduct traveler, which had just completed erection of the south and north approaches, then erected the remainder of the arch deck starting at the north and working continuously across the span. The arch travelers were dismantled by this deck traveler. The concrete roadway and walk were laid in four passes, three for the roadway and one for the walk starting at the north and progressing across the arch span to the south.

The deflections and stresses were calculated on the basis of gross areas and modulus of elasticity 29,000,000. Since the make-up of the arch ribs was very simple, it was anticipated that the actual deflections would be close to the calculated and this conclusion was borne out by observations made frequently in the field.

### Administration

Madigan-Hyland are engineers in charge of construction for the Henry Hudson Parkway Authority, of which Robert Moses, park commissioner of New York City is the sole member. Robinson & Steinman were consulting engineers on the design and Aymar Embury II, consulting architect. The American Bridge Co. had the contract for the superstructure including the concrete deck.

# New Filtration Plant at Springfield, Ill.

HIGH SPEED lime softening and coagulation is an outstanding feature of the new water softening and filtration plant put into operation October 17, 1936, at Springfield, Ill., as part of a municipal improvement program that included the construction of a 7 sq.mi. artificial lake, a power plant, and water treatment plant.

Springfield, for years, had been getting its water supply from the Sangamon River, some seven miles north of the city, where both the pumping plant and power plant were located. With the need for increased water, the city created a new source of supply by damming Sugar Creek about three miles south of the city. The lake thus formed has a shore line of 70 miles and impounds 21 billion gallons of water; this reservoir is the largest artificial body of water in Illinois.

The new water softening and filtration plant has a nominal capacity of 12 m.g.d., subject to 25 per cent overload and is so designed that the ultimate capacity can be increased 300 per cent by the addition of necessary basin and filter structures. Raw water entering the plant has a hardness of 160 p.p.m. which is reduced to about 75 p.p.m.

Charles H. Spaulding, general superintendent of the City Water, Light and Power Department of Springfield conducted the research on the conditioning phase of treatment which led to an economical design of coagulation and settling tanks. The process develops a high concentration of lime floc with water which makes sedimentation rapid and secures better results in an hour and a half and than was secured in six hours at the old plant.

Lime is unloaded by means of a pneumatic system, and gravimetric slakers and feeders are used. Recarbonation, using stack gas from the power plant, completes the softening process. Other features of the water purification plant are air conditioning for the laboratory, in order to eliminate dust and excess moisture in the summer time; electric heating for the entire plant using off peak service; and an attractive architectural treatment of the building both inside and out. In the lobby, an observation well permits inspection of the water leaving the filters.

The municipal electric generating station was placed in operation on May 28 when the first 12,500 kva. turbine generator went into service at the Lakeside plant. The ultimate capacity of the plant will be 62,500 kva. and bids have already been received for the second of the three units that will ultimately be operated.

The $2,500,000 lake and reservoir project was financed by a bond issue to be retired out of earnings by the water department. To help finance the $1,125,000 power plant, $500,000 filtration plant, and $158,000 water main around the lake, the city received in grants and loans from the government a net figure of $1,300,000.

Burns & McDonnell Engineering Co., Kansas City, Mo., were the engineers of both plants and for the lake project. Architectural treatment of the building was executed by Carl T. Meyer, Springfield. Willis J. Spaulding is Commissioner of Public Property.

# Secondary Treatment of Sewage In An Artificial Lake

**Efficient biological purification of settled sewage from a 300,000-gal. disposal plant is provided by a 14-acre lake in which natural reaeration processes supply oxygen at the rate of 50 lb. per acre per day during the winter season**

By F. E. Giesecke and P. J. A. Zeller
*Director and Research Associate, respectively, Engineering Experiment Station Texas Agricultural & Mechanical College College Station, Texas.*

EXPERIMENTAL WORK concerned with the natural purification of sewage in ponds and lakes, carried out in 1930 at Texas A & M College, led to the general conclusion that it was practicable to provide secondary treatment of settled sewage in open bodies of water, and that in such a system of purification, oxygen would be supplied by the lake at the rate of about 74 lb. per acre per day.

Shortly after these studies had been completed it became necessary to enlarge the college sewage treatment plant. In making plans for an improved disposal system it was decided to construct a lake of sufficient size, to provide biological secondary treatment for the entire volume of settled sewage and to serve as a large-scale laboratory for the study of this type of sewage treatment.

## Factors governing design

For the design of the lake it was assumed that (1) the maximum sewage flow would be 300,000 gallons per day; (2) the sewage, after passing through plain sedimentation tanks, would have a biological oxygen demand of 170 parts per million; (3) the lake would supply oxygen at the rate of 70 lb. per acre per day, and (4) the effluent from the lake should have a biological oxygen demand not exceeding 25 parts per million.

On this basis, the lake should have an area of about 5.25 acres. To provide a factor of safety, it was decided to construct a lake having an area of about 10 acres, or about twice as large as that calculated. This was fortunate, since it was later found that the effluent from the new primary treatment plant had a B.O.D. of 375 p.p.m. instead of 170, as had been expected. Consequently, even a 10-acre lake was not quite so large as it should have been, according to the original small-scale, experimental determinations.

The new lake was completed early in the fall of 1933 and settled sewage was discharged into it beginning October 5, without having waited until the lake was well filled with diluting rain water. Very soon thereafter some defects developed in the operation of the primary treatment system and it is probable that large quantities of partly digested sludge, as well as of raw sewage, were discharged into the lake. As a result, the water lost its dissolved oxygen, the fish died, and finally the lake had to be drained after only one and a half month's operation. To guard against a recurrence of the necessity for draining the lake, a bypass was built so that if the primary treatment plant again becomes disabled the raw sewage can be passed around the lake and discharged into the creek below the dam. Before refilling the lake the weir of the spillway was raised 15 in. so as to increase the area from 10 to 14 acres and its volume to about 73 acre-ft. (23.79 m.g.) The lake was then allowed to fill fairly well with rain water, and sewage was discharged into it beginning in April, 1934. Since then the lake has been in continuous successful operation.

When the college authorities cooperated with the federal government in the drought relief program beginning in June, 1934, 60 to 100 head of cattle per day were killed in the college abattoir and the liquid waste was discharged into the sanitary sewer. As a result, the B.O.D. of the settled sewage entering the lake was increased from 375 to 475 p.p.m. Despite this increased load the lake functioned satisfactorily although it did not become entirely filled with water to its rated capacity until some months later.

## Operating plan

The lake covering an area of about 14 acres was formed by building a small dam in the bed of a dry creek. There is no dilution other than that resulting from the rainfall on a comparatively small drainage area.

The entire volume of settled sewage emanating from A & M College, about 300,000 gallons per day, is received by this lake. The sewage may be classified as strong domestic; after sedimentation it has a B.O.D. averaging 370 p.p.m. and a suspended solids content of about 125 p.p.m. The calculated retention period in the lake is 108 days.

Settled sewage is discharged intermittently by a dosing tank into a surge tank from which it flows by gravity through a clay pipe into the lake. The clay pipe is laid along the bottom of the lake and is constructed so that the sewage is discharged through a series of 1-inch openings cut through the top of the clay pipe (Fig. 4); no sewage enters the lake closer than 75 ft. from the shore. The discharge from the dosing tank has been accurately calibrated, and the number of discharges are automatically recorded. This makes it possible to determine the volume of settled sewage discharged into the lake during any desired period of time.

The effluent is taken from the bottom of the lake through a 4-in. pipe about 20 ft. below the surface, and it is discharged through five spray nozzles (Fig. 4) into a dry creek. To determine the volume of the effluent withdrawn, the five spray nozzles (provided with a glass tube to measure the pressure head) and the 4-in. discharge pipe were calibrated for various heads. With the aid of this calibration and the facilities for reading the pressure heads on the spray nozzles, volume determinations were easily made.

The difference between the volume of the influent and that of the effluent during any period of time is equivalent to the amount of the liquid lost by the lake through seepage and evaporation during that period, provided the level of the lake remained constant. Since the lake is constructed in a bed of dense clay and since no seepage has ever appeared on the lower side of the dam, it was safe to assume, therefore, that the entire difference between the volume of the influent and that of the effluent is the quantity of water lost by evaporation, if the lake level remained constant. The evaporation from the lake

FIG. 1—**ARTIFICIAL LAKE** of 14 acres constructed for the secondary treatment of settled sewage. The settling tanks can be seen in the upper left corner; the dam is shown at the right side of the lake.

during January and February was about 22.2 per cent, and during July and August about 54.5 per cent of the influent.

To determine the influence of the lake level on calculations relating to evaporation, a hook gauge was installed so that variations in water level could be accurately measured. Surveys were also made of the shore line of the lake to establish elevation marks so that variations of the volume of the water in the lake resulting from changes in the surface level could be accurately determined.

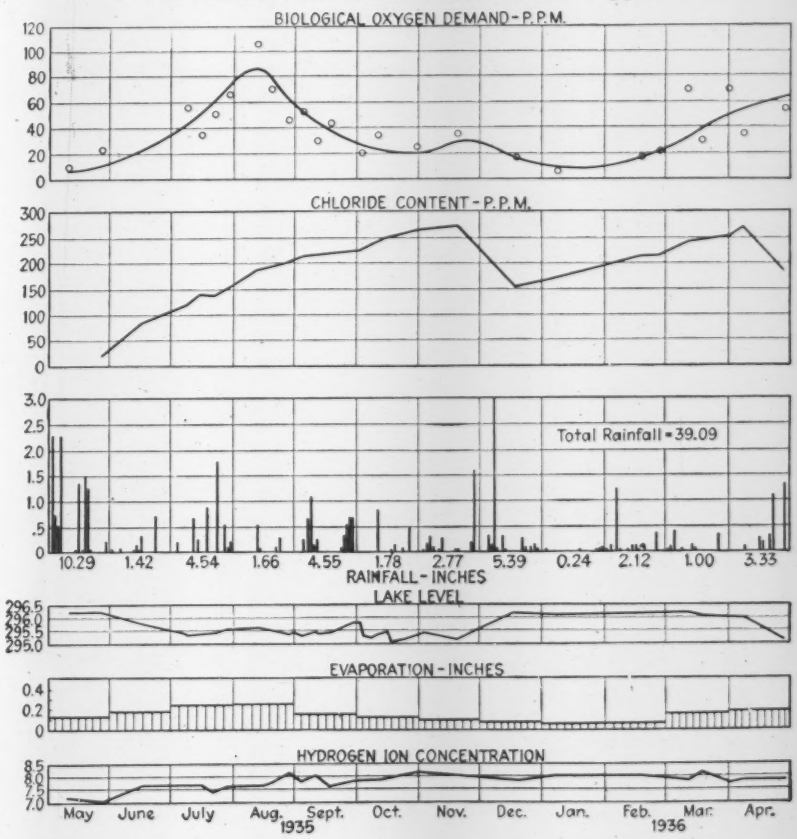
When the volumes of the influent, the effluent, and the evaporation losses are known, it is necessary only to determine the B.O.D. of the influent and of the effluent at frequent time intervals in order to obtain an accurate picture of the operation of the lake as a plant for the secondary treatment of sewage. A graphical presentation of the results obtained, along with other pertinent data, is shown in Fig. 2.

## Operating results

*B. O. D. Changes*—At the beginning of the 12-months period for which results are shown (Fig. 2) the B.O.D. was about 9 p.p.m.; the reason for this low figure was that recent rains had served to provide a considerable amount of dilution water. From the middle of May to the middle of August the B.O.D. increased from 9 to 105 p.p.m. From then on it decreased to about 20 p.p.m. and remained practically constant at that level until the middle of March when it began to increase to reach 60 p.p.m. by the end of April.

The probable cause for the increase in B.O.D. during the first three months and the decrease after that time was the heavy rains at the beginning of the period which not only removed the polluted water from the lake but also removed the oxygen-supplying flora. Since settled sewage was continually discharged into the lake the B.O.D. continued to increase until the flora had been re-established in sufficient quantity to provide the necessary oxygen to

FIG. 2—**CHEMICAL AND PHYSICAL CHANGES** that took place in the lake during a one-year period of operation. The B.O.D. of the settled sewage after passing through the lake was reduced from an average of 370 to 37 p.p.m.



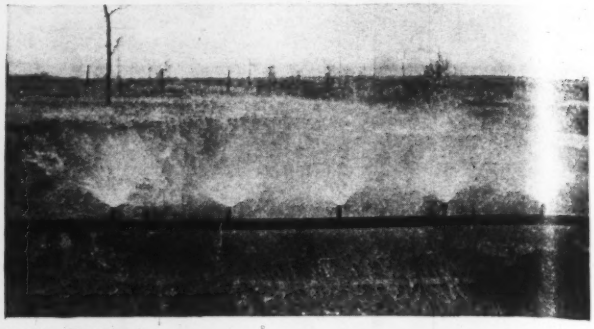


**FIG. 4—INFLUENT** and effluent piping arrangements. The view at the left, taken when the lake was dewatered, shows how the sewage is discharged from the inlet pipe through 1-in. holes. The effluent is drawn-off from the bottom of the lake and aerated by means of spray nozzles (right) before it flows into a nearby dry creek.

establish a condition of equilibrium.

The reduction in B.O.D. from August to November might possibly be considered the result of dilution from rains. But this was not the case as will be shown by a study of the chloride-content and rainfall diagrams (Fig. 2) discussed below.

*Chloride Content*—The college water supply contains about 300 p.p.m. of chlorides. Since a considerable portion of the water discharged in the lake evaporates, the chloride content necessarily increases until heavy rains produce a sufficient dilution. At the beginning of the period the chloride content was negligible because the heavy rains had practically replaced the settled sewage in the lake with pure water. From the middle of May to the middle of November the analysis for chloride content showed a cumulative increase. The B.O.D., on the other hand, increased to a peak early in August, and then decreased rapidly to a fairly uniform level for about five months. This indicates that the decrease in B.O.D. from August to November must have been the result of oxidation and not the result of dilution.

*Rainfall*—It is evident from the diagram that heavy rains fell in the early part of May, which contributed in producing the reduction in B.O.D. and in chloride content referred to above; the heavy rains in November and December produced a similar reduction in these quantities. The total rainfall for the 12-months period, was 39.09 in. and this is in close agreement with a 44-yr. average of 38.49 in.

*Lake Level*—The elevation of the lake spillway crest is 296.25 ft. above sea level and during the period under discussion the water level varied from El. 296.25 to 294.87. These variations reflected changes in rainfall, evaporation losses, rate of influent, and rate of effluent discharge. The water level was also affected by the occasional opening of the spillway gate, as explained below under mosquito control.

*Changes in pH*—Weekly tests were made to the effluent drawn from a sampling connection before the effluent was sprayed to determine the hydrogen ion concentration. It was found that at the beginning of the period the pH value was only slightly above 7 but within three months it rose to 8 and remained practically constant at that value. This indicated the very satisfactory condition of the purification processes within the lake.

**FIG. 5—SPILLWAY** and drainage gate valve structure for controlling the lake level at Texas A & M College.

## Evaporation losses

The evaporation diagram shown in Fig. 2 is the mean evaporation for the respective months according to the official records of the Texas Agricultural Experiment Station. These observations were made on water in a tank 6 ft. in diameter and 2 ft. deep set in the ground within a mile of the lake. The evaporation from the lake however, is somewhat greater than that from a vessel 6 ft. in diameter because the wind, in sweeping over the lake, produces waves which present a larger surface for evaporation than a corresponding level surface.

Several calculations were later made to determine the actual evaporation from the lake, assuming that the loss of water by seepage is negligible. The results were found to be higher than those shown in the diagram. For example: In the 7-day period from July 15 to 22, the influent was 1,502,400 gal. and the effluent, 789,600 gal. leaving a difference of 712,800 gal. During the same period, the lake level fell from El. 295.465 to El. 295.442 or 0.023 ft.; the average lake area during this period was 13.59 acres. The corresponding loss of water, therefore, was 132,300 gal. and the total loss by evaporation was 845,100 gal. This is equivalent to 0.328 in. per day as compared with the mean of 0.224 for July shown in the diagram, and with a maximum of 0.306 in the official records.

This close agreement indicates that the method of determining the quantities of influent and effluent and the variation in lake level was sufficiently accurate for the purposes of this investigation.

## Reaeration observations

The oxygen supplied by the lake is obtained partly by direct absorption from the air in contact with the surface of the water and partly by the life processes of the flora within the lake. The latter is by far the more prolific source of oxygen. Since the growth of the flora depends on sunlight, more oxygen is produced during the day than during the night and the quantity of oxygen dissolved in the water is greater in the afternoon than in the morning. Since the days are longer and the sunlight is brighter during the summer than during the winter, it is obvious that the lake will supply more oxygen per acre per day during the former period.

The rate at which oxygen is supplied by the lake is determined by finding the difference between the oxygen demand of the influent and that of the effluent, when the lake has been in operation a sufficient period of time so that its operation has become stable; this latter condition occurs when the average oxygen demand of the liquid within the lake remains constant. From an examination of the B.O.D. diagram of Fig. 2 it is to be seen that during the 12-months period under consideration the only time during which the operation of the lake was sufficiently stable for such a calculation was the 5-months period between October and February.

During these five months records were taken on 123 days and during these 123 days the influent amounted to 30,070,400 gal. with an average B.O.D. of 370 p.p.m., or 94,630 lb. The effluent amounted to 19,307,900 gal. with the B.O.D. values varying from 6.3 to 35 p.p.m., or a total of 3,370 lb. calculated separately for each weekly period. Accordingly, 91,260 lb. of oxygen demand were satisfied in the lake in 123 days. The area of the lake averaged 14 acres during that period. Consequently, the average rate of reaeration was 52.9 lb. of oxygen per acre per day during the winter season.

In the summer season the rate of reaeration is evidently somewhat higher, but since conditions of rainfall during this period caused a material variation in the operation of the lake, the data available were not sufficient to make an accurate calculation for this particular summer season.

### Cost data

The construction of the artificial lake treatment unit involved an outlay of $7,625 with costs distributed as follows: Land $1,000; 525-ft. dam and appurtenant structures, $5,250; tile pipe influent line, $350; welded steel pipe outlet with spray nozzles, $450; surge tank, $175; bypass duct $400. On a proportional basis, the cost of an installation with a treatment capacity of 1-m.g.d. of sewage per day would be about $25,420.

The operation of the lake requires only occasional cleaning of the screens ahead of the spray nozzle, occasional opening and closing of the drainage valve, and periodic cutting of grass and weeds along the shore of the lake. All of this is easily handled by one man during a small part of his regular time, amounting to about 2 hours daily.

The total cost of operation per year is about as follows:

| | |
|---|---|
| Fixed charges (10% of $7,625.00). | $762 50 |
| Labor, 2 man-hr. daily at 30¢ hr.. | 219 00 |
| Repairs and supplies..... | 50 00 |
| Total ..... | $1,031 50 |

This is equivalent to $9.42 per million gallons, or $4.04 per thousand pounds of B.O.D. removed, and compares very favorably with any other type of secondary treatment.

### A word about mosquito control

There is no difficulty in preventing the breeding of mosquitos in the lake so long as the shores are kept sufficiently free of grass and weeds, so that top minnows can have ready access to all portions of the water near the shore. Occasional opening of the drainage gate valve, to lower the lake level from 6 to 12 in. retards the growth and weeds along the shore line materially. Top minnows are prolific in the lake because of the abundant supply of suitable food material and their presence prevents mosquito breeding without any attention from the operator.

R. T. Richey, a senior student at Texas A & M College made a biological survey of the lake and concluded that three species of fish were especially well adapted to the lake waters, namely: the Plains red shiner (*Notropis lutrensis lutrensis*), the Mississippi black-tailed shiner (*Notropis cercostigma*), and the spotted top minnow (*Fundulus guttatus subsp.*); the first two named are of value as forage fish and the last is of great importance as a mosquito control agent.

### Conclusions

After 2½ years of operating experience it has been found practicable and economical to treat settled sewage in an open body of water without dilution from a running stream and without producing nuisances, either from odors or mosquitoes.

In a treatment operation of this type, lakes will supply oxygen at the rate of 50 lb. per acre per day in localities having climatic conditions similar to those at College Station, Texas.

In starting operation of such a plant it is essential that the lake be practically filled with surface runoff before settled sewage is discharged into it.

The effluent removed from the lake should be taken from the bottom and sprayed before it is discharged.

It is desirable to have a drainage bypass arrangement so that during periods of heavy rains the lake can be washed out and the load on the emergency spillway reduced.

To prevent mosquito breeding it is important to keep the shores of the lake sufficiently clear of grass and weeds so that top water minnows will have access to all portions of the water.

The plant, operated as described, provides efficient secondary treatment at a cost of about $9.42 per million gallons, or about $4.04 per thousand pounds B.O.D. removed.

# Steep Concrete Face On Rockfill Dam in China

**Face of dam articulated because of earthquake hazard and provision is made for relative movement of face and dam**

*THIS UNUSUAL DAM was described in a paper, presented by W. J. Binnie before the Second Congress on Large Dams held in Washington, D. C., in September. It was also described briefly by Harold J. F. Gourley at the convention of the New England Water Works Association in New York City on Sept. 22 (ENR, Oct. 1, 1936, p. 842).*

EDITOR

EARTHQUAKE HAZARDS and a desire for economy in the use of cement led to the adoption of a novel design for a rockfill dam constructed on the Shing Mun River in China. The dam consists of five parts as shown in the accompanying drawing. Its face is a relatively thin diaphragm built of rich reinforced concrete and divided, by vertical, bitumen-filled joints, into segments, so that it can stand a considerable amount of distortion without destruction; a cutoff wall is formed by an extension of the face downward to impervious rock and built of rich unreinforced concrete carefully placed for maximum density. Back of the face of the dam and the cutoff wall is a "thrust block" of low-grade mass concrete, resting on the surface rock and designed to transmit the water pressure to the rockfill behind. Finally, in order to insure a uniform transmission of pressure between the thrust block and the rockfill, a wedge of dry sand is inserted between these elements.

The dam is located on the Shing Mun River, about 12 miles from Kowloon, which is separated from Hongkong by an arm of the sea about a mile wide. The dam forms the Jubilee reservoir which provides an additional supply of water to both these communities and is placed across a narrow gorge, so that, although its height above streambed level is 275 ft., the length is only 695 ft. at top water level.

The rather unusual design was adopted primarily because it was believed that, if an impermeable water face were provided, lean concrete could be used for the other concrete portions of the dam. Since, in China, where labor is cheap, the cost of the cement forms a large part of the cost of concrete in place, considerable economy would be effected by reducing the cement content in the thrust block.

However, the introduction of two mixtures of concrete would give rise to differential movement, due to the fact that the temperature rise is different in a rich and a lean concrete. Thus, in the Shing Mun dam, there was a difference of 29 deg. F. between the temperature rise in the face of the dam and in the thrust block. It was necessary, therefore, to make provision for relative movement of the face and the thrust block.

Also, it was necessary to provide for contraction and expansion in the face by inserting vertical joints.

It was believed that the region where the dam was built was liable to earthquakes; hence it was considered inadvisable to support the impermeable face by a concrete gravity section. The thrust block and rockfill type of construction was therefore adopted instead.

The Shing Mun dam is founded on a fine grain biotite-hornblende granite which has been subjected to a surface decomposition, in some places to depths of 70 to 80 ft. The surface rock is permeable to water, but the underlying granite is very impermeable and will withstand a crushing load of 2,000 tons per sq.ft.

### The cutoff wall

The excavation for the foundation of the cutoff wall was carried down until solid gray granite, free from all but minor faults and those generally filled with impermeable material, was encountered. Exploratory bore holes were then put down to test the impermeability of the rock below the trench; the few open joints found were grouted.

The gorge has a width of about 40 ft. at streambed level, increasing to 120 ft. at El. 453; the cutoff wall was carried up to that level in the gorge, to form the water face.

The cement used was not too finely ground, in order to give a moderate rise in temperature when setting. With the mix used (600 lb. or 6¼ bags of cement per cubuic yard;) a maximum temperature rise of 53 deg. F. was observed. The only natural sand available was a rounded, coarse, sea sand; this was mixed with sand artificially produced from crushed granite in proportions experimentally determined to give the greatest impermeability.

All concrete was placed in thin layers and vibrated with pneumatic tools to release entrained air. Above ground level the concrete was brought up in lifts of 20 ft. at a time, as shown below. It was poured in alternate blocks about 25 ft. wide, bounded together by keyways, in order to minimize contraction as the heat was dissipated.

In order to prevent percolation along the construction joints, strips of copper 0.1 in. thick and 24 in. wide were inserted at the horizontal junction between adjacent lifts. A vertical joint, of greased paper is provided to separate the back of the cutoff wall above ground level from the thrust block.

### The water face

The water face is a reinforced, articulated structure divided into vertical panels 25 ft. wide. Each panel was brought up in lifts of 20 ft. in one operation. The reinforcement consists of horizontal hooked rods wired to verticals, so arranged as to form a grid which could be lowered into the form as one unit.

The face is 6 ft. thick at El. 453, reduces to 3 ft. at El. 573, and continues at that thickness to the top. The general face batter is 1 in 3.4 up to El. 573, and 1 in 3.72 above that elevation. It will be seen that the face batter stops at two-thirds of the height of each lift; the face is vertical above that level so as to form a step 2 ft. in width at the junction of successive lifts which supported the forms for the next lift.

The panels were placed alternately. The outer forms were supported by latticed girders of the queen-post type, the foot of the post being inserted in a groove in the step below. The top of the queen post extended above the form and was tied back to the thrust block; temporary timber struts were inserted to give the correct width and were removed as the concrete was brought up.

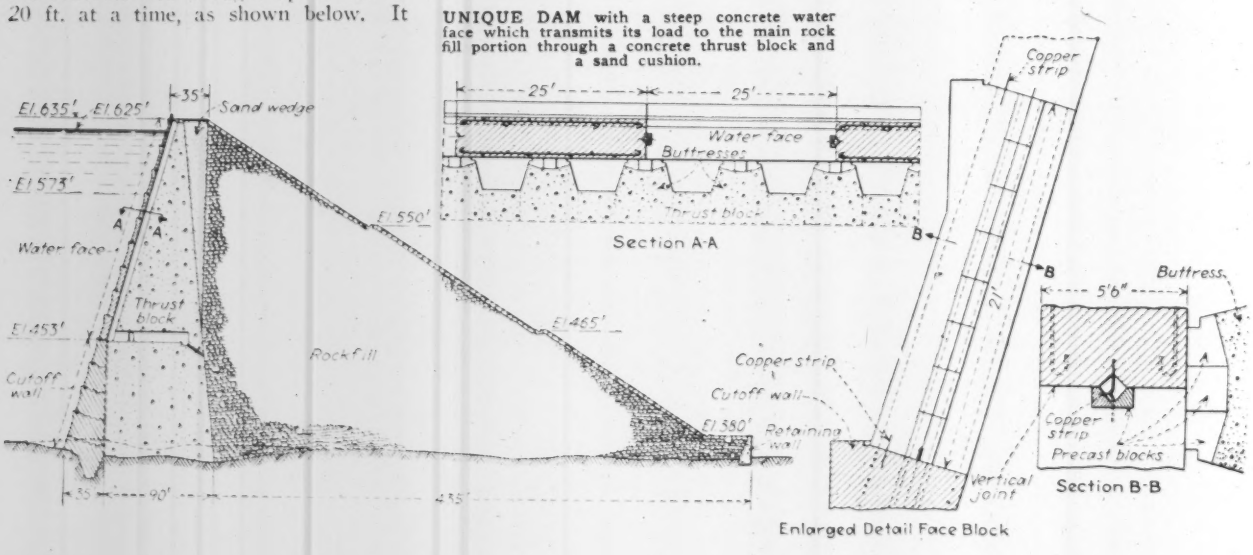
The water face at the back rests, against buttresses protruding from the thrust block, being separated therefrom by a layer of bitumen sheeting 0.2 in. thick, forming a sliding joint. The concrete was poured directly against this bitumen sheeting; timber forms were inserted between the buttresses, housed in grooves formed in the molded concrete blocks with which the buttresses were faced.

Bitumen sheeting was placed in the vertical joints between each panel, and also in the joints where the panels butt against the solid portion of the face at the edges of the dam. The bottom of each panel was separated from the cutoff wall on which it rests by a strip of similar sheeting. Percolation of water through this joint was prevented by a copper strip built into the solid concrete below and carried up into the water face; the width of the strip varied from 2 ft. to 1 ft., according to the water pressure.

A similar copper strip was also provided at the top of each 20 ft. lift. No bitumen was placed between the lifts. The object of inserting this bitumen sheeting at the base of each panel was to permit slight rotation about a horizontal axis without injury to the diaphragm should some slight relative movement of the thrust block take place.

The vertical joints between the panels, which are shown in detail on section *AB*, below, are waterproofed by copper strips kinked in the middle, varying in width from 27 in. to 19 in. The copper strips extend upward in 20 ft. lifts, spliced by an overlap of 7 in. in which small copper bolts are used. A void is formed around the kinks in the copper strips at all vertical joints by inserting small precast concrete blocks. After the face was completed, the void was filled with a mixture of bitumen and granite dust. Two precast blocks are used, one on each side of the copper strip, the column of blocks being brought up to full height before the concrete was poured. The joint between the copper strip and the precast block is made with portland cement mortar, except at the lowest pair of blocks. There an opening is made so

**UNIQUE DAM** with a steep concrete water face which transmits its load to the main rock fill portion through a concrete thrust block and a sand cushion.



that the void can be cleaned out with compressed air before filling.

The concrete mix used in the water face contained 690 lb. (7⅛ bags) of cement per cubic yard, with a maximum size of aggregate of 1½ in. After the forms were stripped, the concrete was kept moist for a minimum period of three weeks by means of water sprays.

### Thrust block design

The foundations of the thrust block were not taken down to impermeable rock, since the decomposed rock on which this part of the structure rests is capable of standing a pressure of about 500 tons per square foot. The width at the top is 8 ft. 6 in., measured over the buttresses, and the back of the thrust block slopes downward at an inclination of 9 on 1. The buttresses are spaced 12.5 ft. apart, leaving inspection pits behind the diaphragm, which can be entered either from the top of the dam or from galleries communicating with the bottom of each inspection pit. These buttresses are faced with molded concrete blocks in order to secure exact alignment of the faces against which the water face rests. The concrete mix contained 300 lb. (3⅛ bags) of cement per cubic yard; large stones were embedded in the concrete. The average rise in temperature observed was 24 deg. F.

The rockfill is hand packed. It was brought up in 2-ft. layers inclined at a slope of 1 on 12 toward the thrust block. The downstream slope of the fill is 1 on 1½, and is broken by two berms having a width of 10 ft. The toe is formed as a platform 37 ft. wide supported by a retaining wall, through which the rockfill drain passes.

The base of the rockfill rests on boulders in the bottom of the gorge and on the soil covering the rock at the sides; all loose or spongy material was removed so as to leave firm ground.

The upstream face of the rockfill, where it is not in contact with the back of the thrust block, is built as a vertical rubble wall in mortar, leaving a triangular space for the sand wedge. Drainage tubes, spaced 10 ft. apart, filled with small pieces of granite retained by screens, pass through this face to give adequate provision for drainage of the sand wedge. The rockfill will be covered with a diamond pattern surface of squared granite blocks when settlement has ceased.

### Sand wedge distributes pressure

The sand wedge fills the triangular space between the thrust block and the rockfill. Its purpose is to avoid point contact and to insure that the water pressure is transmitted from the thrust block should movement of the latter take place. The surface of the wedge is protected from rain by concrete paving. The grains of the sand are well rounded and coarse so as to drain readily should water enter. It was considered essential that the sand should be as nearly dry as possible under all circumstances.

# Sutro Weir Investigations Furnish Discharge Coefficients

**Working values of discharge coefficients on eleven proportional flow weirs are given in nomographic chart and quantities are noted on rating curves**

By E. Soucek, H. E. Howe and F. T. Mavis,
*Research Assistant Engineers, and Associate Director, respectively, Iowa Institute of Hydraulic Research, Iowa City, Ia.*

TESTS OF ELEVEN Sutro proportional flow weirs recently completed at the Iowa Institute of Hydraulic Research indicate that there is a close relationship between the coefficient of discharge and the geometrical proportions of the weir. This relationship is shown in the form of a nomographic chart (Fig. 1) representing discharge coefficients for all weirs tested. The mathematical relationship between discharge and the head above a certain minimum level is linear for the Sutro weir.

In Fig. 1 is shown the general shape of this type of weir, and Table 1 gives values from which the coordinates of the curved portion can be computed, when two controlling dimensions, $a$ and $b$, are known.

### Theory and use of weir

The theory of the Sutro weir was published by E. A. Pratt in *Engineering News*, p. 462, Aug. 27, 1914. For heads greater than $a$ the rate of discharge is directly proportional to the quantity $(h - a/3)$. Practically, the relation between head and discharge is given by

$$Q = ca^{\frac{1}{2}}\, b\, \sqrt{2g\,(h - a/3)} \quad (1)$$

in which $Q$ = the discharge in cu.ft. per sec., $c$ = a coefficient ranging from 0.60 to 0.63 depending upon the dimensions of the weir, $a$ and $b$ are ruling dimensions defining the weir as shown in Fig. 1, and $h$ is the head above the level of the crest in feet.

Using as a basis the elementary principles of flow of water over weirs, it can be shown that if the rate of discharge is to be directly proportional to the elevation of water surface above a datum $a/3$ above the crest, the weir must be bounded by a curve defined by the relation:

$$x/b = 1 - 2/\pi \text{ arc tan } \sqrt{y/a} \quad (2)$$

in which $x$, $y$, $a$, and $b$ are the dimensions shown in Fig. 1.

Weirs with a linear head-discharge relations are particularly adapted for use as controls for grit-chamber outlets, controls for float-regulated dosing devices and flow meters. As control outlets proportional head weirs make it possible to maintain automatically a nearly constant velocity through a chamber with quite large variations in flow. As controls for float-regulated chemical dosing devices, proportional head weirs eliminate the necessity for varying pitch screws, cams, or other eccentrics which must be used with rectangular weirs or other controls. The change in elevation of a float in the head-water pool above a proportional head weir is directly pro-

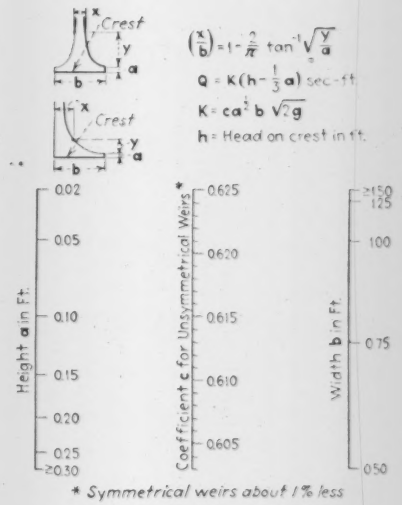



**FIG. 1—DISCHARGE COEFFICIENTS** of Sutro weirs for various combinations of ruling dimensions $a$ and $b$

**TABLE 1—VALUES OF y/a AND x/b, RELATED BY THE EQUATION $x/b = 1 - 2/\pi$ ARCTAN $\sqrt{y/a}$**

| y/a | x/b | y/a | x/b | y/a | x/b |
|---|---|---|---|---|---|
| 0.1 | 0.805 | 1.0 | 0.500 | 10 | 0.195 |
| 0.2 | 0.732 | 2.0 | 0.392 | 12 | 0.179 |
| 0.3 | 0.681 | 3.0 | 0.333 | 14 | 0.166 |
| 0.4 | 0.641 | 4.0 | 0.295 | 16 | 0.156 |
| 0.5 | 0.608 | 5.0 | 0.268 | 18 | 0.147 |
| 0.6 | 0.580 | 6.0 | 0.247 | 20 | 0.140 |
| 0.7 | 0.556 | 7.0 | 0.230 | 25 | 0.126 |
| 0.8 | 0.536 | 8.0 | 0.216 | 30 | 0.115 |
| 0.9 | 0.517 | 9.0 | 0.205 | | |

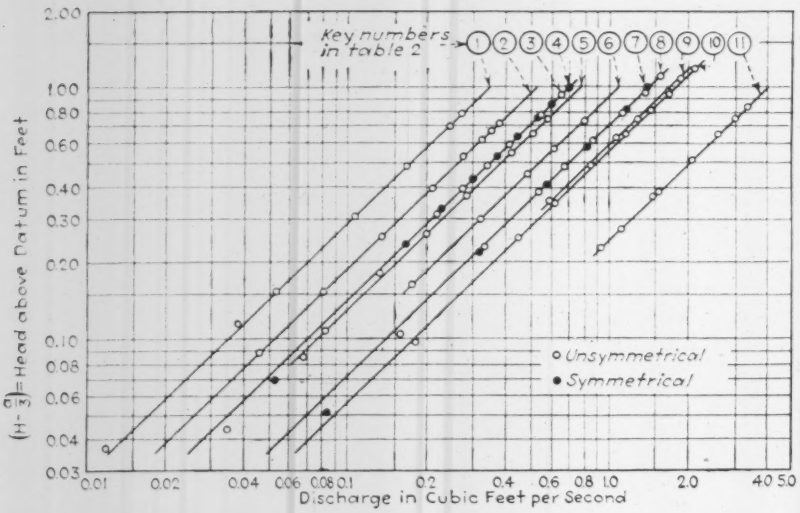



FIG. 2—RATINGS OF eleven Sutro weirs for various combinations of the ruling dimensions *a* and *b*

portional to the change in quantity of water being discharged.

As devices for measuring flow proportional head weirs may be used to advantage in conjunction with recording gages. The flow is a linear function of the head so the mean discharge for any period can be obtained directly from the average head. Further, the sensitivity of the proportional flow weir is constant over a wide range.

In Table 2 is shown the ruling dimensions of all weirs tested and the coefficient *c* for each weir. Weir plates were made of 18-gauge galvanized sheet iron. The boundaries of each opening were carefully marked with a fine scratch awl. The opening was then cut out roughly with a jig saw and accurately filed to the required shape. The plates were securely braced on the downstream side to prevent any possibility of distortion in shape.

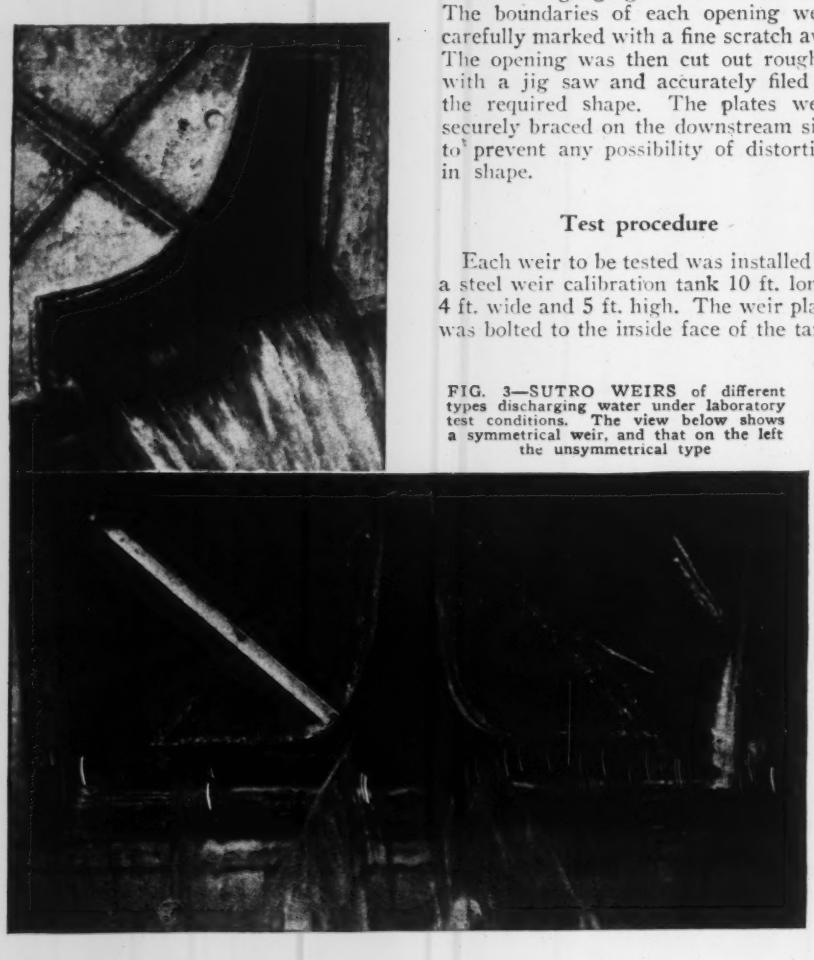


FIG. 3—SUTRO WEIRS of different types discharging water under laboratory test conditions. The view below shows a symmetrical weir, and that on the left the unsymmetrical type

## Test procedure

Each weir to be tested was installed in a steel weir calibration tank 10 ft. long, 4 ft. wide and 5 ft. high. The weir plate was bolted to the inside face of the tank with countersunk flat-head machine bolts and all rough edges in the approaches and connections were carefully smoothed with modeling clay. In each test the weir crest was located at a point approximately 42 in. above the floor of the tank.

Heads on the weir were measured by a hook gage and rates of discharge were measured by weighing and timing. At least two sets of observations were made for each test, and the apparent rates of flow in each case differed by less than 1 per cent.

TABLE 2—SUMMARY OF TEST DATA AND COMPARISON OF OBSERVED COEFFICIENTS WITH THOSE GIVEN BY FIG. 1

| (1) | (2) | (3) | (4) | (5) | (6) | (7) |
|---|---|---|---|---|---|---|
| | | | | c | c | |
| No. | a | b | k | (observed) | (from Fig. 1) | Remarks |
| 1 | 0.02 ft. | 0.50 ft. | 0.348 | 0.614 | 0.614 | |
| 2 | 0.02 | 0.75 | 0.527 | 0.619 | 0.619 | |
| 3 | 0.02 | 1.00 | 0.708 | 0.624 | 0.623 | |
| 4 | 0.02 | 1.00 | 0.696 | 0.614 | ..... | symmetrical |
| 5 | 0.10 | 0.50 | 0.774 | 0.610 | 0.609 | |
| 6 | 0.20 | 0.50 | 1.082 | 0.603 | 0.605 | |
| 7 | 0.02 | 2.00 | 1.401 | 0.618 | ..... | symmetrical |
| 8 | 0.02 | 2.00 | 1.417 | 0.625 | 0.625 | |
| 9 | 0.50 | 0.50 | 1.715 | 0.605 | 0.603 | |
| 10 | 0.02 | 2.50 | 1.772 | 0.625 | 0.625 | |
| 11 | 0.30 | 1.50 | 4.030 | 0.612 | 0.614 | |

## Results obtained

A graphical summary of the data obtained from these tests is shown in Fig. 2. The ordinates show values of $(h - a/3)$, or the elevation of the water surface above the datum fixed by the mathematical analysis. The abscissas show the corresponding rate of discharge in cubic feet per second. The number on each curve refers to the number of the weir described in detail in Table 2.

In Fig. 1 the analysis of test data on unsymmetrical, contracted weirs is shown in the form of a nomographic chart. The scales *a* and *b* show, respectively, the ruling dimensions *a* and *b* of the unsymmetrical, contracted Sutro weirs shown in Fig. 2. The center scale, *c*, gives values of the coefficient, *c* in Formula (1). Any straight line which intersects these three scales defines by the points of intersection of *a*, *b*, and *c*, respectively, the corresponding values of the ruling dimensions *a* and *b*, and the weir coefficient *c* as determined on the basis of the Iowa tests.

## Application of chart

The nomographic chart applies directly to unsymmetrical, contracted Sutro weirs. For symmetrical sharp-edged Sutro weirs the coefficient determined from Fig. 1 should be multiplied by 0.99.

Columns 5 and 6 of Table 2 shows a comparison of the observed coefficient, *c*, determined by large scale plotting of test data, and the corresponding coefficient computed by Fig. 1. The maximum discrepancy between observed and computed coefficients is 0.3 per cent of their mean.

# Freeway Design Principles Applied On a Rural Road

## Norris dam highway gives freeway service on a small traffic road through a rural territory

By Earle S. Draper
*Director Land Planning and Housing Division Tennessee Valley Authority, Knoxville, Tenn.*

*FREEWAY TRAVEL is a road service commonly held unavailable except on highways of dense traffic in metropolitan areas. There are, as a matter of fact few metropolitan main traffic arteries that are so designed. So when the Tennessee Valley Authority undertook a freeway to connect its great Norris Dam development with the Tennessee road system to the north and south it originated a new idea for an essentially rural highway of moderate traffic. Except for a short section of concrete, the surfacing is of moderate quality. These characteristics, together with the fact that the road accommodated heavy freight traffic to Norris Dam during the construction period, justify full consideration of the TVA freeway.* —EDITOR.

The principal feature that distinguishes the Norris Freeway from other highways is the width of its right-of-way and control of its borders. This right-of-way has a minimum width of 250 ft. throughout the road's entire 21-mile length, and it was acquired as an outright purchase in fee simple rather than through the more usual procedure of obtaining an easement for passage over property to which the owner retains title. Furthermore, the right of access at any point—the prerogative of properties abutting on an ordinary highway—is restricted to points agreed on at the time of right-of-way purchase, and they are strictly limited in number and location.

### Freight road first

The heavy-duty section of the freeway was built first and is a 4.1-mi. concrete pavement extending from the railhead at Coal Creek to the damsite (Fig. 1). In this stretch the heavy tonnage to be hauled was the determining factor in the structural design of the pavement. This has a 10-8-10-in. cross-section of reinforced concrete 22 ft. wide—the extra width being established by the requirements for heavy trucks and the transportation of large pieces of equipment. The concrete was poured in two lanes with transverse expansion joints at 60-ft. intervals.

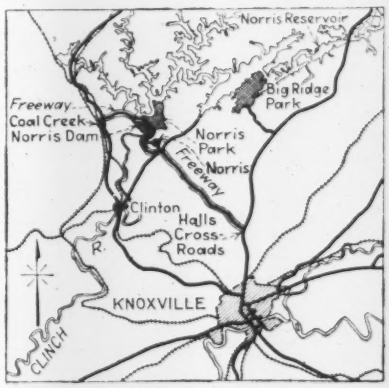



**FIG. 1—FREEWAY LOCATION with respect to Norris Dam and the Tennessee state road system**

To facilitate the beginning of construction on Norris Dam it was necessary that this section of the pavement should be placed as rapidly as possible and record-making time was made. The contract was let to W. W. Boxley & Co., of Roanoke, Va., on Nov. 1, 1933, grading was begun Nov. 7 and paving was started Nov. 29; the first lane of pavement was finished Jan. 23, 1934, and the second lane was completed on Feb. 15. Thus, the heavy-duty section was rushed to completion in only 69 winter working days. Due to the rugged nature of the terrain, the roadbed required a considerable number of deep cuts and fills which involved the moving of more than 210,000 cu. yd. of excavation, including some 20,000 cu. yd. of solid rock. The pavement required more than 55,000 sq. yd. of concrete, reinforced with 435,000 lb. of steel.

As the concrete paving immediately followed the grading operation, it was necessary to compact the subgrade so as to avoid excessive shrinkage or settlement. This was accomplished by depositing the fill in 12-in. layers each of which was thoroughly compacted by ten-ton rollers. (For details of the construction of this section of the freeway see *Engineering News-Record*, Feb. 22, 1934, p. 254.)

That the construction methods employed were adequate is evidenced by the fact that during the intensive dam construction period, extending over more than two years, the freeway gave no signs of structural failure. Such cracks as have developed are infrequent and of a minor nature and there is no evidence of surface wear nor subsidence of the roadbed. The tests to which the pavement has been subjected were severe. The tonnage transported to date amounts to 300,000 tons, or nearly 1½ million ton-miles. The largest-size single piece hauled was a bridge girder 105 ft. long. The heaviest single load was a transformer weighing 55½ tons, which was hauled on a 10-ton truck.

The actual cost of maintenance of the freeway for the fiscal year July 1, 1935 to June 30, 1936 was $11,682, or less than 5c. per sq. yd. of actual road surface, and it covers maintenance costs for the entire 250-ft. width of the right-of-way including keeping ditches and drainage system open, grass cutting twice a year, as well as the repair and upkeep of the riding surface. Of this maintenance cost, about 50 per cent is for wages and salaries and about 25 per cent for materials. The remaining 25 per cent is apportioned to miscellaneous supplies, transporting working crews, repairs and depreciation on equipment, medical attendance for service-connected injuries, and a pro-rata share of the TVA's divisional and general administrative expense.

### General service connection

The immediate need for the section of the freeway extending from Norris Dam to the existing connection to Knoxville at Hall's Cross Roads (Fig 1) was augmented by the construction of the new town of Norris, which lies about four miles south of the dam on the way to Knoxville. Originally the shortest usable route between Knoxville and Norris was by roundabout highways with a distance of about 26 miles between downtown Knoxville and the town of Norris. The freeway reduces this distance to about 20 miles.

**FIG. 2—HEAVY-FREIGHT SECTION** of TVA freeway was paved with a 22 ft. slab of reinforced concrete of 10:8:10-in. section; a 250 ft. right-of-way was carefully handled to preserve natural roadside beauty

Frequent field trips to the dam and townsite by members of TVA's Knoxville personnel, and the need for a road for hauling materials from the Knoxville area to the field justified the immediate construction of this stretch of the freeway. Furthermore, by utilizing Norris Dam as a river crossing, the freeway provides the most direct route between Knoxville and the north, and forms an important link in the access system to the recreational areas now developed or to be developed in the Norris region. Near Knoxville the freeway connects with paved thoroughfares extending from the city toward the north.

This section of the freeway, extending from Norris Dam to Hall's Cross Roads is a 20-ft. macadam roadway consisting of a 6-in. crushed stone base treated with an 80-lb. per sq. yd. mixed-in-place bituminous wearing surface. The base was placed as a rolled macadam surface and allowed to compact for several months before the top treatment was applied.

Although for the purposes of the foregoing description the freeway is referred to as consisting of two sections, surfaced with different materials, the freeway is really a continuous roadway connected by the causeway across the crest of Norris Dam, and both sections embody the same basic principles of freeway design.

### Location by airviews

In designing the freeway, an engineering procedure somewhat different from the usual practice was employed. In this method, stereoscopic studies of air photos were used to supplement planetable strip topography, thus combining the advantages of both sources of data. As a preliminary step, stereoscopic studies of air photos were used to determine possible locations for the route. These preliminary lines were then briefly inspected in the field for the purpose of selecting the most suitable one for development. The proposed line was then laid out on the air photos which were sent to the field to be used for guidance in running a location traverse. The base line data obtained in the field were then returned to the office and computed and plotted on coordinated matched plane table sheets. Again in the field, planetable strip topography was taken along the traverse at a scale of 50 ft. to the inch, and the sheets returned to the office, where the preliminary lines and profiles were studied and tentatively approved. The approved preliminary line was then rough staked in the field by planetable method and inspected. After minor revisions had been made, the final line was computed in the office and sent to the field in looseleaf form for final staking. Thus, all computations of alignment and grades were carried out as office procedure rather than field work. Due to the fact that the designer in the office was enabled to visualize the line as an entity, rather than as broken or detached stretches pieced together in the field, the method proved to be both practical and time saving and produced an excellent alignment.

The engineering design of the freeway was conditioned by the rough terrain to which it was necessary to adapt the requirements for an efficient, safe, economical, and attractive rural roadway based upon the application of the most suitable highway practice to the problems growing out of the high-speed traffic of today. To this end, sweeping curves were combined with superelevation to produce a safe roadway extending through semi-mountainous country. The alignment was thoroughly studied so as to take full advantage of the existing conditions of the natural ground and thus avoid excessive grading; at the same time, however, hazards such as blind spots and abrupt vertical curves or dips were eliminated. Thus, alignment was carefully adjusted so as to combine the requirement for safe, rapid transportation with pleasing esthetic qualities so that the effect is that of a scenic highway attractively fitted to the natural landscape. Curves of at least 500-ft. radius, and maximum gradients of seven per cent were established as the desirable controlling standards. By the use of flat slopes on cuts and embankments—varying from 1 on 2 to as low as 1 on 4 or 5 and by rounding these slopes, erosion has been reduced, maintenance diminished, safety promoted, and, at the same time, a pleasing appearance has been gained.

In four instances, curves of 300-ft. radius were employed. Except at the immediate approaches to the dam, where the use of the crest of the dam as a causeway over the Clinch River imposed the surmounting of a difficult terrain, there are no other short radius curves along the freeway.

There are four short stretches along the freeway where the grades exceed seven per cent. One of 600 ft. and another of 800 ft. have grades of 7½ per cent; there is one 500-ft. stretch having a grade of 8 per cent and one 10 per cent grade 300 ft. long.

Theoretical or abstract discussions of safety mean little unless supported by facts and figures. In this respect the

freeway has made an excellent showing. Since June 1934 the TVA force of guides and guards has been responsible for the patrol of the greater portion of the freeway, and, up to Sept. 1, 1936, their records contain a total of only fifteen reportable accidents with an aggregate damage of less than $300 to the cars involved.

These figures gain added significance when the volume of traffic is considered. During the 1934-35 construction period the freeway was used by TVA buses in transporting more than a million passengers to and from their work on the dam. Since June, 1934, almost 800,000 visitors have registered at the damsite and it is estimated that at least half as many more were not recorded. All of these visitors necessarily used the freeway or portions of it to reach the dam and, in most cases, the visit involved round-trips. In addition to this there is the normal local traffic accruing to the freeway and the thousands of trips made by TVA employees between the Knoxville offices and the dam or the town of Norris. Tennessee highway commission traffic counts give an average of about 100 cars per hour using the freeway during a normal 12-hour period. Thus, while the freeway's volume of traffic is not comparable with that of some of the major highways, such as those serving metropolitan areas, nevertheless it is considerably greater than the average for the country's main highway system. However, the rural surroundings and the slight amount of cross traffic on the occasional intersecting minor roads did not justify the immediate expenditure of funds for grade separation as would have been considered necessary in a more densely populated area.

## Roadside development

The country traversed by the freeway has considerable scenic merit—rugged, wooded stretches interspersed by farmland and offering frequent and extensive views of the surrounding mountains. Therefore, for esthetic reasons, the rural character of the freeway was maintained and any introduction of a discordant urban parkway theme was avoided. To this end, formal agreements with the farmers from whom the land was bought entitled them to continue to cultivate land lying within the outer portions of the right-of-way wherever this can be done without impairing traffic safety. In return for this privilege the adjacent property owners agree not to permit the erection of billboards upon the remainder of their farmland or its buildings and not to build any business structures adjacent to the freeway unless proper approval is first obtained. These agreements of course form important supplements to the use regulations which govern the right-of-way itself. This method of control has proven highly successful in eliminating unauthorized access roads and roadside nuisances such as billboards, stands, shacks, and other undesirable developments along the freeway and its borderland.

To further the naturalistic effect of the freeway's setting, the thinning of existing trees and undergrowth was resorted to only under the necessity of eliminating traffic hazards or for the purpose of opening up a vista. As the greater portion of the freeway traverses forest and woodland, roadside planting was considered superfluous, but where necessary to effect good composition, native shrubs and trees have been set out. Close-growing plants and ground covers have been established and maintained on slopes and banks where needed to check erosion; berm ditches, check dams and other slope protection were also provided. Several informal picnic grounds developed along the freeway have proven extremely popular.

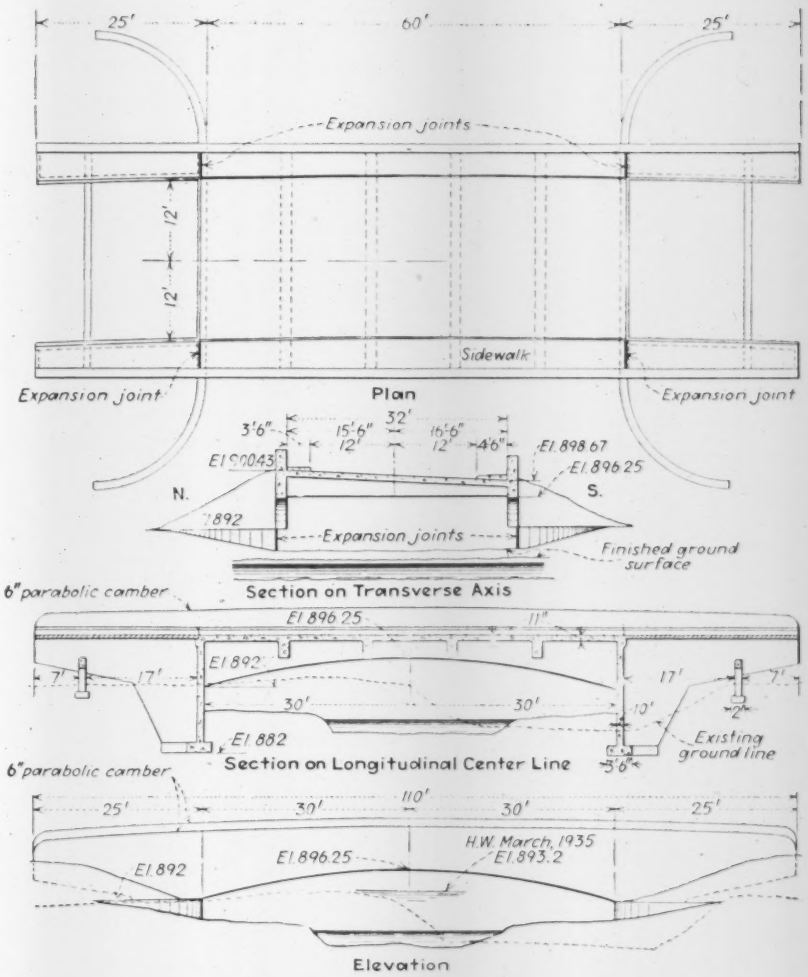



**FIG. 3—BRIDGES** along the freeway are notable for their curved abutment design and unusual clear width

**FIG. 4—A TYPICAL SECTION** of the concrete freeway brings out the gentle slopes that prevail throughout its 22 miles of length of both concrete and bituminous surfacing

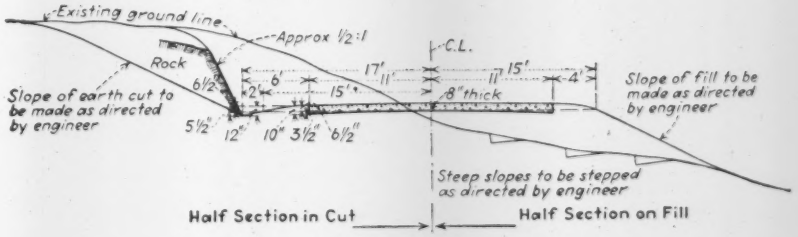


FIG. 5—TYPICAL BOX CULVERT along the freeway; note the rounded corners and slightly cambered top. A metal plate guard rail is used along the roadway

A modern type of steel-plate guard rail is used. The degree of care which entered into the esthetic design of the freeway is reflected in the appearance of the bridges and culverts which span the numerous brooks and streams which occur along the route. Although the designs differ radically from the usual highway structures, they are neither elaborate in appearance nor unduly expensive to build. The aim was to combine a functional architectural treatment with sound engineering practice. The principal outward characteristics of the freeway structures are the broad unadorned surfaces and the introduction of curvature in the headwalls and wingwalls. Simplicity rather than embellishment is the keynote; the chief reliance for effect consists of pleasing lines and good proportions. Particular attention was given to the flow of line in bridge rails and walls as viewed by passing traffic. Furthermore, each structure was designed to fit the individual requirements of its particular location, thus adapting it to the site as well as to its purpose. All of these structures are of concrete, and economy of design was gained by the use of rigid frame construction wherever possible.

The freeway demonstration of roadside improvement is already producing results. Without any urging by the TVA, garden clubs and business men's associations of the locality have enlisted the aid of the state highway department in protection and planting of the portion of the Tennessee state highway No. 33, that connects the freeway with Knoxville.

This may be truly called a rural freeway in that it adapts the freeway principle of highway construction to rural conditions and moderate traffic use. Excessive costs of land purchase or construction of protective or architectural features unnecessary to the situation were avoided.

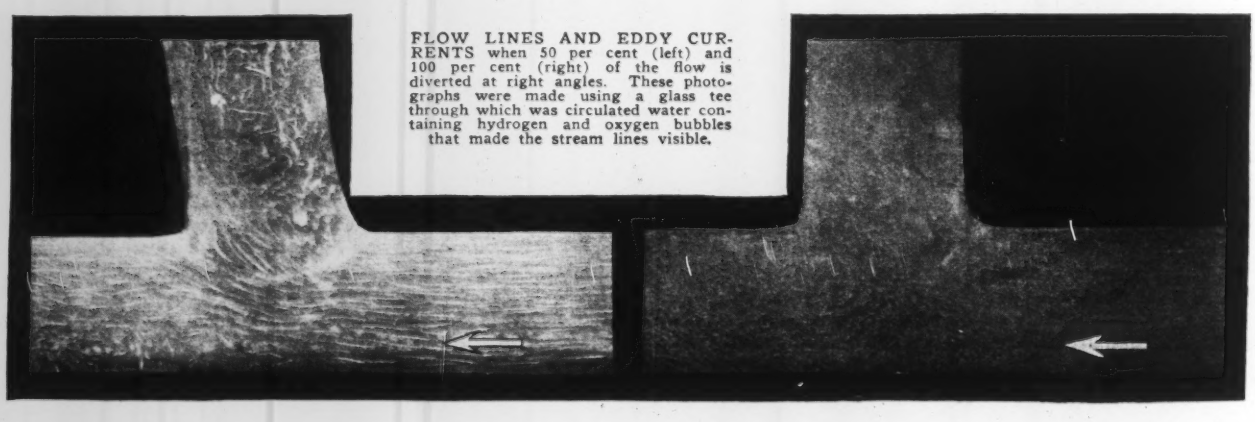
# Flow Turbulence Analyzed by Means of Photography

IN A STUDY of the loss of head in cast iron tees made at Texas A. & M. College, College Station, Texas, and reported in Bulletin No. 41, photography, was effectively employed as an aid in checking experimental data and conclusions regarding lines of flow and turbulence. A glass tee with an inside diameter of 1 in. was substituted for the cast iron tee in the piping system so that the flow of water could be observed. In order to make the stream-line visible, small bubbles of gas (hydrogen and oxygen) were produced electrolytically in the water stream, which had been acidulated by the addition of sulphuric acid.

The bubbles passed down the stream and into a horizontal plane of light produced by an arc light throwing its beams through a narrow horizontal slot in a metal shield between the source of light and the tee. The bubbles, in passing through this plane of light, reflected it very much as a minute mirror, thus enabling the eye to visualize the line of flow.

In order to photograph these lines of flow, sufficient illumination was produced by substituting photo-flash lamps for the arc light. The accompanying photographs are illustrative of the results obtained.

At the left is a picture of the eddies and lines of flow when approximately 50 per cent of the water is diverted at right angles. From this it can be seen that most of the water which passes straight through the run of the tee does so with little disturbance, for there is only a slight eddying of the current in the run on the side opposite the branch outlet. The water which is diverted, however, impinges against the side of the outlet, causing a violent eddy formation which passes down the stream with a helical movement, finally straightening out again in parallel lines of flow. The photograph on the right shows the lines of flow when 100 per cent of the water entering the tee is diverted. From this it was observed that, due to large amount of turbulence and eddy currents, the loss of head would be larger than that in an ordinary elbow.



FLOW LINES AND EDDY CURRENTS when 50 per cent (left) and 100 per cent (right) of the flow is diverted at right angles. These photographs were made using a glass tee through which was circulated water containing hydrogen and oxygen bubbles that made the stream lines visible.

CURRENT HIGHWAY PROBLEMS—VII

# Intercity Arterial Roads

**Growing traffic forecasts the need of trunkline road service between principal cities involving a measure of express road design**

By E. C. Lawton

*Assistant Commissioner (Construction), New York Division of Highways, Albany, N. Y.*

*BETWEEN the neighborhood main road and the through express highway there is, in the recent thinking of highway engineers, need of an intermediate classification which may be designated as intercity arterial roads. These roads are unlike express roads in excluding general traffic and like them in providing particularly for heavy, fast, long-distance travel. In America we have a few of these arterial roads and the beginnings of others as the Boston-Worcester, Chicago-Milwaukee and Kansas City-Topeka trunk roads, but no general state or broader system of such highways. The following article is a consideration of such a system for a state. Its author is one of the keenest thinkers and writers on the problem of the arterial highway and his consideration of the engineering requirements of location, design, construction and transport value is of penetrating interest.* —EDITOR.

IT SEEMS to be common practice to use the term intercity main-road system and the term primary or state roads as synonymous. This is rather unfortunate. In New York state which has 13,000 miles of improved state roads, only a small percentage of them might be classed as intercity main roads. In general, we consider those routes carrying from about 3,000 up to 15,000 or more vehicles in twelve hours, as the primary system. At present it embraces about 3,000 miles. For the purposes of this article I have interpreted an intercity main-road system to include the upper traffic range of the primary system. These roads should also form a logical connection between large centers of population, either directly or supplemented by means of spur connections.

While in New York no official designation or classification of an intercity main-road system has been made, it should naturally form a connection (two routes) from the New York city area to the vicinity of Albany, thence west to or near larger cities such as Schenectady, Utica, Syracuse, Rochester, Buffalo, and west to the Pennsylvania state line. One or two roads of this type will be necessary between New York city and the Boston Post Road region; a similar one from Albany eastward to the Massachusetts line, and a third from New York city westward toward Liberty, N. Y. Additions may be necessary but no further attempt at selection or refinement is included herein. Such a system would be within 5 mi. of about 80 per cent of the population of the state. It can reasonably be estimated at about 900 miles, which is about 30 per cent of the primary system and 7 per cent of the total system.

A rational approach to a problem of this magnitude naturally must take into consideration its probable cost, the element of time, the saving of human life and the saving in cost of highway transportation. As construction costs are dependent upon an assumed typical section we may now discuss generally what such a typical section and general layout should be.

## The problem broadly

In a discussion of any road system, particularly an intercity main-road system, a distinction should be made between the terms pavement design and highway design. Pavement design has particular reference to the structural design of the pavement itself, such as type, width, depth, and super-elevation of curves. Highway design has to do primarily with problems of location, sight distance, alignment, grade, signs and a consideration of possible regulation of highway traffic. Most of the elements of highway design are determined by ordinary rules of highway safety while pavement design has to do primarily with highway economics, although there is a relationship between the two.

The first problem of major importance is location. In this state the most available locations along valley routes are now occupied by improved highways serving the public. In general, therefore, the locations of intercity main roads will be on new location and in rolling or rugged country. In congested areas where a new location is impracticable, highways will be elevated or depressed to insure a reasonable speed and to avoid accidents. New locations will be more flexible in the hands of the designer in selecting appropriate alignment, grade, sight distance, etc., than would be possible along existing highways. This is for the reason that land values are relatively high along existing highways and in urban areas there are frequent street intersections, irregular and uncertain movement of pedestrians, and traffic may be unduly impeded by numerous stop and go signals.

While roads of this type will be designed primarily for heavy traffic, it is obvious that they will attract a large volume of passenger vehicles, due to advanced highway design. The resulting traffic, therefore, will be of the mixed type, capable of making different rates of speed, and to obtain maximum safety, highway design will require four lanes of pavement with so-called island construction between the pairs of lanes. This type of road should have a minimum sight distance (four lanes) of 500 ft., preferably 1,000 ft., maximum grades of about 5 per cent, minimum radius of curve of 1,500 ft., preferably 2,500 ft., limited access, preferably at intervals of about four miles and cross traffic separated by highway grade separations; it should not lead directly through congested parts of cities, but should have spur connections. A minimum number of warning signs or signals should be used but there should be shown in a conspicuous manner alongside the pavement the maximum speeds permissible in different areas, commonly known as speed zones.

In New York state, with 60 cities and over 550 villages, in which rolling and rugged country predominates, and with only a limited number of so-called water-level routes in congested areas, the possibility of constructing an intercity main-road system has some serious and formidable limitations. As an example of such limitations, let us discuss the requirements of such roads.

## Island separations

Safety requires that travel in opposite directions be separated; that cross traffic be eliminated by the use of grade

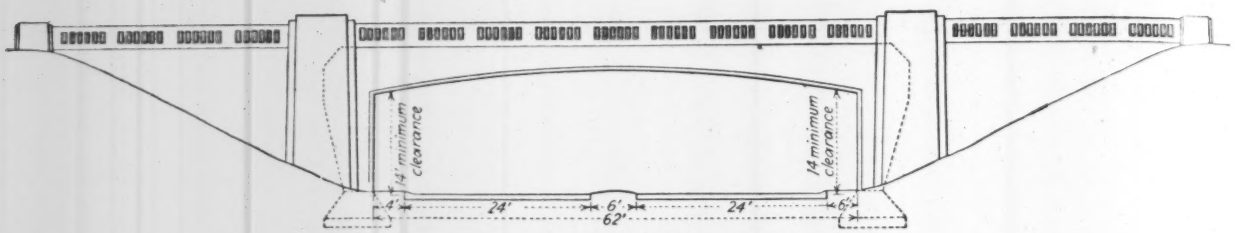



FIG. 1—CONCRETE RIGID FRAME bridge as used to carry intersecting roads over wide intercity arterial highways

separation structures and that side or entering traffic be allowed at infrequent intervals. Assuming these as proper fundamentals, there are, of course, related problems of great importance, each affecting one or more of the fundamentals. The more important may be listed as proper width of pavement lanes and avenues, proper limiting sight distances, grades, curves, signs, etc., as influenced by the kind of terrain in which the highway may be situated.

Island construction is commonly accepted as the most effective method of separating traffic and promoting safety. The width of island may vary from a minimum of 3 or 4 ft. to a maximum of 20 ft. or more, dependent upon the area (suburban or rural), in which the road may be located. It is not necessary that islands be curbed but it would seem desirable that they be protected by a narrow slope paving. This narrow band of slope paving, if of concrete, can be made of white cement which will function as a distinct traffic marker.

The islands may be developed as grass plots or planted with low shrubbery to improve their appearance and discourage traffic from driving upon them. In open country, however, if they are not planted with shrubbery, but used as grass plots, they afford a convenient place for depositing snow removed from the pavement in winter. This is of some moment on wide pavements of this type, especially from the standpoint of cost of snow removal.

Each pavement avenue should be constructed, if of concrete, with an integral curb 4 in. high on the outside and bevelled towards the center at an angle of 45 deg. When an integral curb is used the outside pavement lane should be 13 ft. in width and the inside pavement lane 11 ft. in width. The curb really functions as a part of the drainage system and if constructed of white cement will also function as a pavement marker. Such a curb will eliminate the need of deep ditches which are a menace to safety. A curb of this height, with a bevelled side, can be driven over in cases of emergency, such as flat tires, etc.

A 6-ft. berm should be built back of the curb (in both cut and fill sections) to provide a space for depositing snow. It will reduce ditch- and shoulder-maintenance cost, serve for emergency parking and finally, it will lend itself to appropriate landscaping or sodding.

If the pavement is of concrete it should be at least 9 in. thick and adequately reinforced. In the immediate vicinity of densely populated areas and over doubtful subsoils, it should be 10 in. thick. If there are doubts of the carrying capacity of the subsoil, it should be stabilized with 6 in. of gravel or similar material. Other types of pavement are equally adapted to traffic of this character, if they are of sufficient depth and carrying capacity.

The transverse crown of the pavement may vary from a minimum of $\frac{1}{8}$ in. per foot on the inside lane to a maximum of $\frac{1}{4}$ in. per foot on the outside lane. The latter slope increases the water carrying capacity of the curb above described and is desirable for this reason *only*. Any greater crown is objectionable from a traffic standpoint. The minimum crown of $\frac{1}{8}$ in. per foot will function satisfactorily in rolling country but is not well adapted to flat grades, say, up to one or two per cent, where curbing is used.

Joints in concrete pavement are objectionable and should be as far apart as good practice will permit. In New York state present spacing is about 90 ft. with a distinct trend towards a greater spacing. Joints should be of the load transfer type, for both shear and flexure. A trench about 1 ft. wide and deep filled with porous material should be provided as a weep outlet through the shoulder at the outside and of each joint space. This weep will provide an outlet for water which tends to collect at the joint due to pumping action of the pavement slab and the presence of moisture in the subgrade.

Joint filler should be absolutely of the non-extruding type to avoid impact shocks. It should be somewhat compressible and should have sufficient recovery to prevent foreign matter from getting into the joint space and restrict expansion of the slab.

## Road grade separations

Grade separation structures for reasons of economy and safety should be as far apart as possible. A desirable minimum would be four miles, but in congested areas they will be more frequent, possibly a minimum of one or two miles apart. There are several economical types of acceptable and pleasing structures that may be used to good advantage. Those, however, with a minimum depth between the clearance line and floor line will be the most economical as, with a given grade, the access roads may be shorter and the amount of fill kept to a minimum.

A discussion of appropriate grade separation structure and the highway facilities to be provided in connection therewith is rather difficult without definite knowledge as to what percentage of an intercity main-road system would be located in open country and what percentage in urban and metropolitan areas. For reasons before assigned, i.e., that new location outside of urban districts may predominate, a design may anticipate a reasonable choice of essentials and then of desirable features. The essential ones are to provide a vertical clearance of 14 ft. and to permit traffic to leave the highway without delay and compel it virtually to stop before entering the highway; avoid sharp curves on high fills and do not compel traffic to make a sharp turn on either ascending or descending steep grade; acquire ample right of way to permit proper development of these facilities and also to prevent encroachments that might obstruct a clear view of other vehicles. These elements are essential for reasons of safety.

For reasons of economy, a location that permits partial depression of one highway and partial elevation of the other, at the point of crossing, should be selected. This will insure an approximate balancing of excavation and embankment, will permit access roads of reasonable length and will require a minimum amount of right of way. Neither road, however, should be depressed to a point that will create blind spots at points of entry and exit. Also, a shallow floor system will result in a minimum rise or fall between the two intersecting pavements, thus reducing grades and shortening the length of access roads.

With these general assumptions, a structure with a clear span of 62 ft. will be required to accommodate two pavement avenues each 24 ft. wide, a center island 6 ft. wide and two 4-ft. sidewalks. While the island in open country may be 10 ft. or more in width, it may taper down to 4 ft. at overhead structures to reduce the cost of the structures.

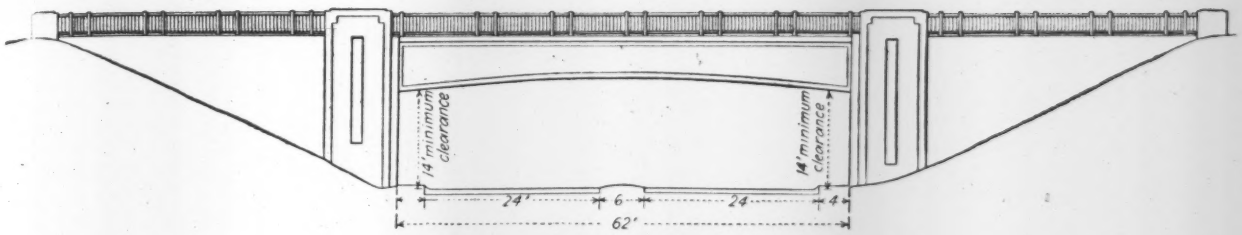



FIG. 2—STEEL GIRDER overhead crossings of arterial highways are given attractiveness by curved lower chords and tasteful abutments and hand rails

The two types of structures most commonly used by the New York division of highways are the rigid frame arch (Fig. 1) and the ordinary girder type (Fig. 2). The rigid frame type is generally of concrete which lends itself readily to aesthetic treatment. Such treatment may be a rubble masonry facing or the concrete may be scored to resemble coursed masonry. If the girder type with ordinary abutment is used, a curved lower chord is more pleasing in appearance. The steel can be painted with a color that harmonizes with the abutments and surroundings. There is only a nominal difference in cost between the girder type and the rigid frame type where the skew is not great; as the skew increases, however, the girder or steel rigid frame type becomes more economical.

At important intersections with much interchange of traffic the construction of inner loops within the outer access roads is quite essential. They have been shown dotted in Fig. 3 to indicate that the time of constructing them may be deferred until traffic increases to a point where it becomes necessary to do so. If the stage method of construction is adopted, the outer access roads will carry two-way traffic and should be two lanes in width. When the inner loops are constructed, the outer access roads will function for one-way travel and signs should be displayed to inform traffic. One of the splayed entrances on the main artery and on the main highway should then be discontinued and barred to traffic. In a complete cloverleaf design including outer access roads and inner loops each of them will be for one-way traffic. However, the curves of 100-ft. radius should have a 20-ft. pavement to permit trucks with trailers to make the turn.

## Road-rail grade separations

On an intercity main-road system railroads should be separated from the highway by appropriate overhead or underpass construction, whichever is better adapted to the locality. Highway-railroad grade crossing separation structures are simple as compared with purely highway grade separation structures. This is for the reason that there are no access roads to contend with and without such access roads there is

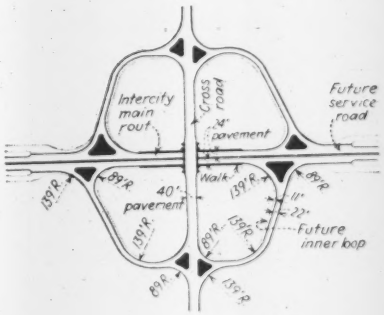



FIG. 3—TYPICAL PLAN for traffic exchange grade separation of arterial highway and an important cross-road

no entering traffic from the side to be guarded against. On new location efforts should be made to obtain an angle of crossing that will permit the use of the steel girder type of construction where railroads are carried over highways. Bridges of this type with concrete masonry abutments lend themselves to a limited amount of esthetic treatment which should harmonize with the general surroundings. Esthetic treatment usually consists of the application of an appropriate color paint on the steelwork and scoring the concrete to resemble coursed masonry. Of course it goes without saying that the same standard of highway facilities such as proper sight distance and appropriate grades apply to the same extent on highway portions of these separations as apply to highway grade separation structures.

The cost of highway-railroad grade separation varies greatly, dependent upon the angle of skew, the number of tracks and whether or not the tracks have to be supported or may be detoured. The number of structures of this type that may be encountered depends entirely upon the terrain and for that reason it is impracticable to make a general estimate covering work of this character.

## Service road problems

In residential, urban or congested areas it may be necessary to construct service roads in addition to the main avenues along the arterial highway. These service roads will collect all local and intersecting traffic and carry it to grade separation structures where it is permitted to enter the arterial highway or the cross road. Service roads should be generally parallel to the main avenues of travel but with no connection between them except in the vicinity of grade separation structures. A portion of the island between the main avenue and the nearest service road may be paved as a cross-over for traffic on the service road desiring to enter the main avenue and proceed in the same direction. If service road traffic wishes to go any other direction it should enter the outer access road and proceed in the normal manner. This separation between the main avenue and the service road may be accomplished by additional islands similar to that above described. The pavement on the service road need not necessarily be of the same depth or carrying capacity as on the main or arterial avenue but it should be borne in mind that it may later become a main artery. If constructed of concrete, I would recommend that it be of the conventional 8-7-8-in. depth of pavement. Other types of pavement of similar carrying capacity are equally adapted to service of this character. In residential regions, other types of surfacing may be considered more appropriate. In this article I do not presume to discuss the relative merits of types of pavement but rather fundamental principles that should govern design.

In congested areas where construction on expensive right of way is unavoidable, the width of safety islands may be decreased and service roads may be built at higher or lower levels than the main avenues. This type of construction is more in the nature of a special problem of detailed design but the same fundamental reasons governing design should be adhered to in order to obtain maximum service and safety at minimum expense.

## Roadside control

Assuming that roads of this type will in general be upon new location, a minimum right of way of about 200 ft., except at separations, should be acquired to permit maximum development. By maximum development is meant a sufficient area within which the state division of highways or similar

appropriate unit may have control over the use of such lands. It should be borne in mind that as traffic increases in volume it may become necessary to construct additional avenues of main-line travel. If the right of way is adequate, this may be easily accomplished by using the original service roads as main avenues and adding another service road if needs require. Also, if the lands originally acquired are not completely utilized by pavement and roadway areas, such acquisition will be justified from the standpoint of safety along the main avenues and the service roads. Such added areas may be used for carrying public utility properties but under such conditions as the central agency may see fit to impose. The state should have control over a sufficient area to prevent developments or the placing of signs that might be inimical to safety and objectionable to the public interest.

### Cost and economics

Estimates of costs must be general in character, on account of the general nature of the subject. Assuming a 24-ft. avenue of 9-in. concrete in each direction, separated by a 10-ft. island in the center and providing a 6-ft. berm back of the 4-in. integral curb and with grade separation structures two miles apart, a reasonable estimate of construction costs may be assumed as $200,000 per mile. In built-up regions with service roads the cost will be increased. The added cost of two service roads, each with 24 ft. of pavement, may reasonably be estimated at $90,000 per mile. The cost of intercity main-road systems could then reasonably be estimated to vary from $200,000 per mile in rural regions to $300,000 per mile in urban regions. It should be pointed out that these estimated costs do not include the cost of right of way or long-span bridges.

The cost of right-of-way will vary materially with the location. In metropolitan areas the cost would be very high and probably have to be purchased on a square foot or frontage basis. In rural areas it would be relatively low and probably could be purchased on an acreage basis. Similarly, the cost of bridges would be dependent on the frequency of crossing large streams. Neither of these items can be reasonably estimated until the approximate location is known. For example, a projected Pelham-Port Chester connection from New York City to the Boston Post Road region, having a length of about 12 miles, is estimated to cost (construction only) $13,500,000. The right-of-way therefor is estimated to cost about $7,000,000. This, of course, is about the most congested region in the state. At the other extreme, it may reasonably be anticipated that the construction costs in rural territory would be about $200,000 per mile and the right-of-way costs about $50,000 per mile. Hence some preliminary knowledge as to the general location selected for highways of this type is essential before attempting to estimate the total cost of such a program.

The importance and need of constructing roads of this character is almost self-evident although it does not follow that they must all be built at once. On the other hand, it is a matter of common knowledge that some parts are essential at this time. It should be pointed out and emphasized that there are many features that are actual savings of public funds. The construction of certain logical portions at this time would make it unnecessary to reconstruct a large mileage of highways now overloaded, inadequate, dangerous or obsolete. The saving in reconstruction could be devoted to intercity travel purposes. The new work would provide both safety and capacity which cannot be obtained on present highways at almost any price. The safety to be obtained has been described above. Now let us discuss capacity.

A recent comparative study by the New York legislative highway survey committee discloses that the estimated 900 miles of intercity system may reasonably expect to carry an average of 9,000 vehicles per day (a maximum was nearer 25,000 vehicles). The resulting annual traffic is about 3,000,000,000 vehicle miles. This is 33 per cent of the total traffic carried by the entire 13,000 miles on the state system on only 7 per cent of such system.

It is improper to place a money value on loss of life or loss due to injuries. The record of 3,500 deaths and 10,000 injuries on all streets and public highways of the state speaks for itself. Neither can it be claimed that roads of this type can save any definite amount of human suffering and loss. On the other hand, it is obvious that they would materially reduce accidents and hazards of motor vehicle travel. From an economic standpoint, if we assume a modest saving in transportation cost of $.01 per vehicle mile, the savings would aggregate $30,000,000 per annum. Over a 10-year period a large part of this system could be completed.

# Railway Bridge Maintenance Discussed by Engineers

**Protecting steel from refrigerator car brine, affect of high-speed trains on bridges and preframing of bridge timbers discussed at Chicago**

VARIOUS ASPECTS of railway bridge construction and maintenance were the subjects passed in review at the 43rd annual meeting of the American Railway Bridge and Building Association, held at Chicago on Oct. 20 to 22. That more frequent inspection of submerged parts of bridge piers and abutments would result in economy by enabling light repairs to forestall more extensive or emergency work, was the theme of a committee report presented by E. C. Neville (Canadian National Railways). Typical examples of foundation failures and repair work formed a large part of the report, and were supplemented in the discussion by statements of individual experience of foundation works. A prominent suggestion was that heavy rip-rap should be placed carefully, instead of being simply dumped in place. For new bridge work, it was suggested that cofferdams of steel sheetpiling should be used and left in place as protection against scour and as an insurance against future repair work under water. Bridge design in relation to maintenance was dealt with in a paper by R. P. Hart (Missouri Pacific Ry.), and the work of the bridge department as affected by the present railway situation was presented by A. R. Wilson (Pennsylvania R. R.) in an address of welcome as president of the American Railway Engineering Association.

### Brine corrosion on bridges

Of special importance relative to maintenance was a report (B. R. Meyers, Chicago & Northwestern R. R.) on the protection of steel bridges from the corrosive effects of brine dripping from refrigerator cars. It has been shown that although the annual excess maintenance cost of bridges and track due to this cause is estimated at $2,350,000, this is not sufficient to warrant the capital cost of equipping the cars with brine-retaining tanks and the necessary fittings. Mechanical cooling apparatus and dry-ice refrigeration do not produce destructive drippings, but these methods are used as yet on only a few cars. However, the

committee foresees increased use of cars of these types. The drippings from cars of coal, ore and other materials have also to be considered.

Of 35 railroads reporting, only eight consider their protective methods effective on open-deck bridges, and these few roads have but small refrigerator-car traffic. Some roads consider that ballasted decks afford the best protection, but the extra cost is not always warranted. Most railroads use heavy coatings, such as special paints, petrolatums, crude oils, bridge cements, and asphalt or tar and their products. In fact, many roads use several different materials. But there is increasing use of petrolatums and road oils, as these contain rust inhibitors and do not require a thorough cleaning of the steel, thus reducing the cost of application. A coating of gas-house tar applied over graphite paint and then covered with sand has stood up well under severe conditions.

A method of steel protection used by the American Bridge Co. begins with allowing the steel to weather for two to six months to loosen the mill scale; after cleaning with power-driven rotary brushes a rust inhibitor is applied, having a phosphoric and soluble chromate base. Over this is a paint primer of zinc chromate-iron oxide pigment with a synthetic-resin base vehicle containing phenol-formaldehyde varnish. The finish coats are of graphite or aluminum paint, with the same type of vehicle as the primer. Zinc-metallic pigment is recommended as a protection against brine, and the increasing use of copper-bearing steel is an aid in reducing corrosion troubles. Brine protection needs to be applied with a frequency depending upon the density of refrigerator-car traffic. A method not yet used to any extent in railway work is "metallizing," or spraying of molten metal on the surface to be protected. In the design of a bridge, an additional cover plate or an extra thickness of cover plates or flange angles increases the weight and cost by only a small amount but adds considerable protection against deterioration by brine droppings, thus becoming an insurance against early need of repairs.

## Bridges and high-speed trains

Bridge maintenance and repair as related to or affected by recent high-speed train developments, was discussed in a committee report (G. E. Boyd, *Railway Engineering and Maintenance*), showing that with higher speeds the vibrations and destructive effects of trains increase more rapidly. This necessitates closer and more frequent inspection, especially on open-deck bridges. In particular, the floor system develops loose rivets and cracks in connections. It is evident that certain details of design must be modified to meet the new conditions resulting from higher speeds, and these conditions apply with the general increase in speeds of passenger and freight trains as well as with the as yet infrequent trains of specially high speeds. The introduction of light-weight passenger cars is also a factor as to easy riding, since slight irregularities may cause an unpleasant bounding motion.

The general conclusion presented was that under conditions of increasing speeds the bridge work will not be very different from that of the present day, except for the very important point that it will have to be done without delaying trains. As speed increases, the disadvantages of speed restrictions increase in greater degree, and obviously it would be highly undesirable to obstruct these especially fast trains by slow orders at various points on account of bridge or track work. For this reason it will be necessary to exercise great care in the planning and execution of such work. For example, where falsework is required it must be designed to carry trains at full speed.

## Preframing for timber bridges

A report on the relative merits of different species of woods for bridges, dealt only with trestles and did not include timber trusses (G. S. Crites, Baltimore & Ohio R. R.). It presented a surprisingly long list of species suitable for piles of temporary structures and where the entire length of the piles will be permanently below groundwater elevation. For temporary work, local availability and economical prices are the governing considerations, with conditions of easy or hard driving as a factor. In general, piles for main-line bridges should be treated.

Structural timbers noted include sills, posts and compression members, caps, stringers and bracing. For braces, a gripping device between the brace and timber was recommended, so that the bolts would have only to hold the brace in position. This is desirable since bolting alone may develop only 10 to 20 per cent of the available strength of the braces. A great variety of timbers for use treated or untreated was reviewed, and it was stated that preservative treatment does not lower the resistance to bending, hardness or compression perpendicular to grain. Flooring for ballasted-deck trestles and for highway bridges over the tracks should be of treated material; this becomes especially economical with the prevalence of rubber-tired highway vehicles, under which the floors will fail by decay rather than by wear. Laminated floors permit the use of cheaper grades of wood, especially when made with a sawtooth surface and covered with a bituminous wearing material.

Dense Douglas fir is the timber now most available for structural timbers, but dense pines, larch, cypress, redwood or hemlock may be available and economical in certain localities. However, Douglas fir is coming more into use, owing to the increasing scarcity and higher cost of select pines and oaks. Where possible to make selection, the men in the field should use the denser timbers for caps, sills and blocking. A limited discussion turned largely on the merits of Douglas fir for different purposes.

Supplementing the above report was that of the committee on development of preframing for timber bridges (O. W. Stevens, Delaware & Hudson R. R.), dealing with the framing of trestle timbers in advance of preservative treatment. While there are difficulties and the practice requires a high degree of accuracy in design and in the measurements for renewal work, the committee pointed out that the steel industry has set an example of how it can be done in its shop fabrication of structural steel. In fact, some railroads now have standard forms for recording field measurements of timber parts for renewal, these being supplemented by framing diagrams which simplify the information for the fabricating plant.

Treated piles are in general use, but as it is not practicable to drive them to exact depths it is useless to preframe or prebore them. But with modern methods of trestle construction there is little necessity for framing, boring or reaming holes in the field. Any such field work will cause trouble unless followed by swabbing with hot preservative and hot sealing compounds in holes before the bolts are applied. A number of roads have built to standard plans open-deck and ballasted trestles more than 1,000 ft. long in which all material, including the ties, was prebored and preframed, and was erected with practically no field boring and framing. A cautionary statement pointed out that as this work is done by bridge carpenters, it is essential for economic results that these men should be trained in the proper handling of treated material. This idea was elaborated in a report on improving the bridge and building organization (W. B. Mackenzie, St. Louis-San Francisco Ry.).

## Buildings and water tanks

Inspection and repair of tanks and towers (R. E. Dove, C., M., St. P. & P. Ry.) were dealt with mainly in regard to wood tanks, but included conical-bottom and flat-bottom steel tanks. In these last, corrosion is generally in the floor, owing to acids and minerals in the sediment or sludge. Cleaning periods depend upon the character of the water, but in general once a year is sufficient for flat-bottom tanks, while those with conical bottoms should be blown out once a week or month, according to whether the water is muddy or clear. Special conditions occur with the use of water-softening processes.

Recent developments in paints for

building and structural use were discussed by F. L. Browne (U. S. Forest Products Laboratory), more especially as to the behavior of paints on frame houses. Then the termite problem as affecting railway structures was reviewed by Harry R. Duncan (C., B. & Q. R. R.). In roofs of various materials the troubles and failures are due more to lack of adequate maintenance than to poor materials or workmanship in original construction, according to a committee report (J. S. Hancock, D., T. & I. R. R.).

Subjects for the 1937 meeting include: (1) Developments in timber trestles; (2) deferred maintenance in relation to painting; (3) maintenance of movable bridges; (4) safety [illegible] use of power-operated tools and equipment; (5) fire protection of structures; (6) water service developments; (7) insulation of railway buildings. [illegible]election of officers resulted as follows: President, E. C. Neville, bridge and building master, Canadian National Railways; secretary, C. A. Lichty, [illegible]hicago.

# Cantilever Bridge Erected From One End

## Wabash Railway Company builds new structure over the Missouri River at St. Charles, eliminating 50-year old bridge

By S. M. Smith
*Engineer of Design, Wabash Railway, St. Louis, Mo.*

DESIGNED to be erected from both ends, a cantilever bridge across the Missouri River at St. Charles has just been completed by the Wabash Railway Company, construction having been carried on entirely from one end. When it became evident that a considerable amount of time would be required to dismantle the erecting equipment and re-erect it on the opposite side, the feasibility of erecting the structure all from one end was investigated. It was found that no change in details would be necessary, except for the removal of some splice plates, and a rearrangement of the temporary steel bents. As a result of this change, it was estimated that a month's time in erection could be saved.

The structure, which went into regular service Oct. 13, replaces one of the old historic landmarks in its section of the country, first constructed in 1871 and rebuilt during the Eighties. The old bridge consists of seven spans some 300 and some 315 ft. in length on high masonry piers having a clearance of 70 ft. in ordinary stages of the river. Four of the spans are of the Whipple double intersection type 40 ft. deep, and three are of the double Warren type 33 ft. in depth. The details of the latter trusses gave more or less trouble as the floorbeams were carried on brackets riveted to the inclined web members. The trusses were originally designed to carry 84½-ton locomotives, but with the increase in traffic and in the size of locomotives, they were strengthened to carry one 210-ton locomotive. While the computations showed that the unit stresses allowable had not been exceeded, the increased loading no doubt contributed to the weakening and ultimate renewal of the bridge.

Early in 1929, surveys were started for a new bridge. The height of the piers of the old bridge and the irregular alignment of the approaches made a relocation advisable. A new crossing was located about one-half mile downstream from the existing line. At the same time it was decided to reduce the ruling grade between St. Louis and Kansas City, located just east of the bridge, necessitating a change of line between Robertson and St. Charles, a distance of 7.2 miles. The line selected shortened the distance about 0.7 mile and eliminated 234 degs. of curvature.

FIG 1—MAIN CANTILEVER span of the new Missouri River crossing of the Wabash Ry. provides a 600 x 45.5 ft. shipping opening.

The War Department required that the railroad company provide a vertical clearance of 45.5 ft. above the 1903 high water mark, one 600 ft. clear main channel and one 406 ft. auxiliary channel, locating the westerly main channel pier about 250 ft. from the west bank. With these restrictions, it was impossible to obtain a symmetrical structure. A cantilevel structure with unequal anchor arms was selected which met all of the requirements, and a simple span was inserted at the east end, to fill out the waterway. The suspended span was made identical with the simple span, thereby obtaining a small amount of duplication in the details. The simple span was made 312 ft. in length, the east anchor arm 429 ft., the channel span 624 ft. and the west anchor arm 273 ft.

### Main bridge details

The structure is designed for single track, using a special loading of two 401-ton locomotives, the equivalent of about Cooper's E-75. The basic unit stresses used were 18,000 lb. per sq. in. for carbon steel and 24,000 lb. per sq. in. for silicon and heat-treated steel. The floor system and all main truss members except one in the simple span are of silicon steel. In detailing the structure, the back-to-back of chord angles was held constant instead of the inside-to-inside of gusset plates as is usual. This simplified the riveting in the chords and enabled the gussets to be cut into the webs of the chords, thus shortening the length of the connecting rivets in addition to saving material. Split H-sections were used wherever possible in the bracing. A star section—two angles with equal legs arranged in the

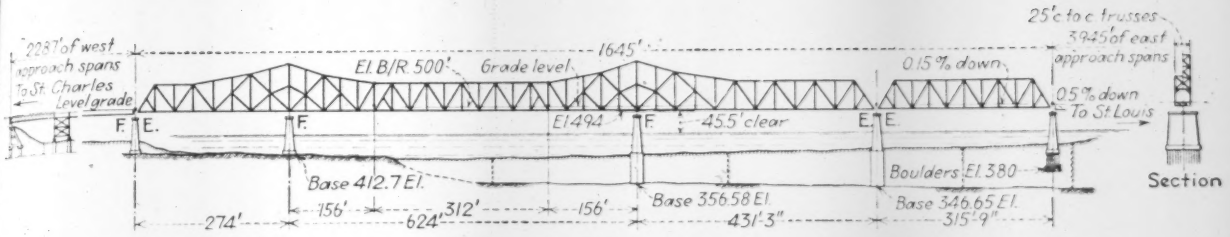



FIG. 2—**UNSYMMETRICAL** anchor spans and a suspended span that is a duplicate of the simple approach span characterize the new Wabash Ry. bridge over the Missouri River at St. Charles, Mo.

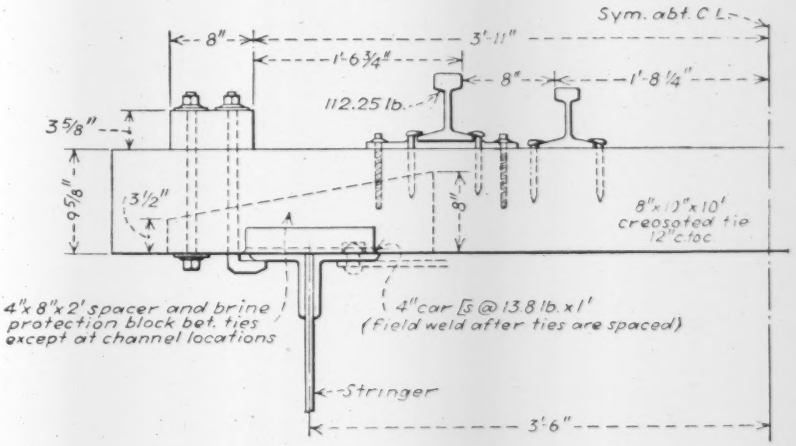



FIG. 3—**DECK SECTION** in which spacer blocks between the ties protects the stringers from brine drippings. Steel channels welded to the stringers replace the blocks at intervals in the main span, serving to fix the ties in position.

shape of a cross or an X—are used in the diagonal bracing of the intermediate portals. The expansion shoes at the end of the anchor arms are designed with segmental rollers and adjustable wedges which permits of ample adjustment. In the short anchor arm, the uplift is taken by eight 4-in. round rods extending 30 ft. down into the pier. The shoes are set on a bush-hammered surface on alternate layers of canvas and red lead paste. The expansion in the 624 ft. span is taken at the east end of the suspended span, the west end being fixed to the west cantilever arm.

The five main piers were constructed during the winter of 1930-31 by the Missouri Valley Bridge and Iron Co. The three intermediate piers are faced with Indiana limestone from the base to a few feet above the high water line. All of the piers rest on bed rock except the easterly one, which is supported on a foundation of concrete piles. On the completion of the piers, no further work was done on the bridge for several years. However, some 1,000,000 cu. yd. of earth were moved on the change of line before all work was suspended.

## Long approaches

The substructure of the east approach consists of 154 concrete pedestals supported on creosoted wood piles. The tops of the pedestals are on the average about 4½ ft. square and are built so that the elevation of the tops are above the high water level, and at the same time parallel to the 0.5 per cent grade in steps, as it were, to produce as much duplication in the superstructure as possible. The east abutment is 38 ft. in height, and is of the spill-through type. It consists of two posts with a beam across the top and is supported against overturning by two inclined members acting as braces. The volume of concrete for an abutment of this height was only 103 cu. yd.

In the west approach substructure, there are 82 concrete pedestals. Elevation of bedrock being only about 30 ft. below the surface of the ground, it was thought that the wood piles used in the east approach would broom in driving. Consideration was then given to the use of steel H-sections, but owing to the irregularity of the surface of the bed rock, these were abandoned as it would be impossible to determine the unit pressure on the rock due to the small area of the pile section. To obviate these difficulties, concrete cylinders were adopted as meeting all of the objections to both the wood and steel piling. The diameters of the cylinders used are 4, 4½ and 5 ft. The west abutment is similar in construction to the east abutment, except that it is 55 ft. high. It is supported on four 5-ft. cylinders and contains 201 cu. yd. of concrete, exclusive of the cylinders.

The superstructure of both approaches is similar in construction, except that the east approach is on a 0.5 per cent grade and the west approach is on a level grade. It consists of a series of a 60-60-30 spans on towers and rocker bents alternately. The columns are all rolled sections, silicon steel, either 24 or 27 in. in depth. All columns are provided with cast iron pedestals, which rest in a bed of iron filings, salamoniac and sulphur. In the rocker bents, the columns are fixed to the bases, being designed to overcome the stresses due to traction and expansion. Silicon steel is used in all girders 60 ft. in length or over. For the sake of appearance and also economy, the 30 ft. spans in the towers were made the same depth as the adjacent 60 ft. spans. To reduce the friction in the expansion bearings at the top of the rocker bents, bearing blocks of structural steel fitted with phosphor-bronze are provided, which can be lubricated from the track level with a pressure gun.

## Erection from one end

The erection of the main spans was started at the east end of the bridge with the simple span, and was carried on by means of an inside traveller supported on the stringers and bottom chords of the trusses. The simple span was erected on four steel bents, each bent being supported on eight steel H-piles.

The east six panels of the east anchor arm were cantilevered out from the simple span to an erection bent, a strut being placed between the bottom chords and a tie between the top chords the latter consisting of eyebars later used in the permanent structure. The channel span was cantilevered out to the mid-point of the suspended span and the four erection bents used on the simple span were re-erected under the west half of the suspended span and the west cantilever arm. In erecting the west anchor arm, only one erection bent was required; when the erection of the west anchor arm had progressed two panels beyond the pier, the westerly bent of the four bents in the channel span was dismantled and re-erected at the third panel point west of the pier. The erection bents in the channel span were erected above the final grade so that when a landing was made on the westerly pier, the end floorbeam would be about 1 ft. low. This was done to

facilitate the erection of the final top chord member. The end floorbeam was temporarily carried on timber blocking and finally jacked to the proper elevation, at the same time lowering the remaining bents in the channel. Then the shoes and the anchorages were placed in the proper position.

### Special deck construction

The details adopted for the deck construction are a modification of the Wabash standard deck, introducing some of the features of the German track construction in an attempt to prevent the creeping of the rails and at the same time to provide some protection against brine drippings on the top flanges of the stringers and the girders. The ties, which are 8 x 10 in. and 10 ft. long, creosoted and prebored, are spaced on 1 ft. centers and are separated by brine blocks, 2 ft. long, 4 in. thick, 4 inches wide at one end and 8 in. at the other. The brine blocks are spiked to the ties with one 8 in. boat spike with the wide end toward the center of the track so that they will drain to the outside. These brine blocks also function as spacer blocks to prevent bunching in the case of a derailment. In each panel in the main spans, six of the brine blocks have been replaced with 4-in. car channels 12 in. long, welded to the stringer. The ties are thus fixed to the stringers and the rail is prevented from running in either direction by rail anchors inserted on the rails between the ties held by the channels. In the approaches, the rail is fixed to the tower spans in the same manner.

### Track characteristics

The main running rail is the new *A.R.E.A.* 112-lb. section, using double shouldered tie plates. The tie plates are attached to the ties by two screw spikes. The inside guard rail is of the same section as the running rail. The outside guard rail is a 4 x 8 in. timber laid flat and not dapped. The deck is held in position by hook bolts in every second tie, the alternate ties being held by a bolt through the guard rail. The ties are not dapped as the top cover plates are run full length, but in the girder spans with rivet heads in the upper flanges, the ties were grooved before treating. All holes in the ties were prebored except those for the hook bolts. After the track was lined into position, these holes were drilled in the field. The outside guard rails were not prebored. It was the original intention to attach the tie plates to the ties before placing, using the template described in the A.R.E.A. Manual, but it was found that any slight irregularity in the location of the tie plates prevented the rail from entering between the shoulders on the plate. The procedure finally adopted was simply to lay the tie plates in position, lay the rail, then spike and finally apply the screw spikes.

### Administration

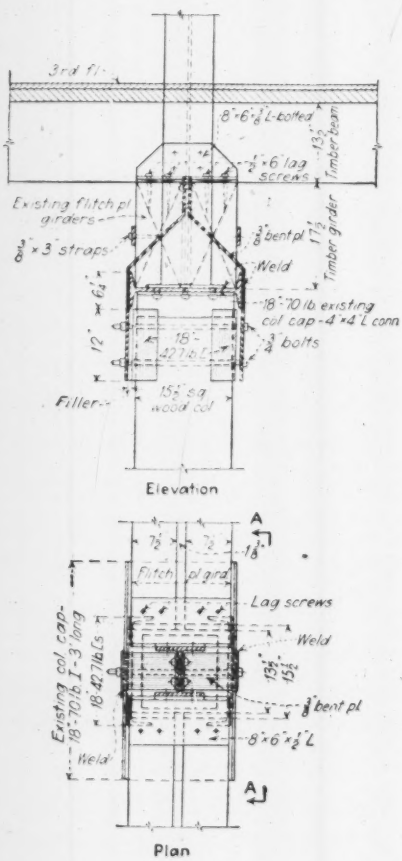
The bridge is 7876 ft. in length and contains 8800 tons of structural steel, 155 tons of reinforcing bars, 5860 yards of concrete, 4061 lin. feet of concrete piles and 64228 lin. ft. of wood piles, exclusive of the five main piers. The total cost of the structure was $1,720,000 of which $1,065,000 was for the superstructure. The Inland Construction Co. was the contractor for the east approach substructure, and Bates & Rogers for the west approach substructure. The contract for the fabrication and erection of the superstructure was let jointly to the American Bridge Co. and the Mt. Vernon Bridge Co. The American Bridge Co. fabricated and erected the approaches and the Mt. Vernon Bridge Co. the main spans. John T. Ellis and Walter Smith were the superintendents for the respective companies.

The work was done under the direction of E. L. Crugar, chief engineer, Wabash Railway Co. and C. S. Johnson, bridge engineer. The structure was designed by the writer, who prepared plans for both the substructure and the superstructure. R. S. Stephens was the resident engineer in charge of the field work. Modjeski, Masters & Case were the consulting engineers, and C. T. Duncan the resident engineer-inspector for the Public Works Administration.

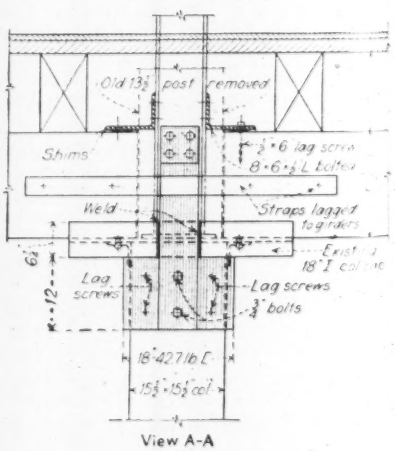
# New Stories of Steel Framing Added to Timber Building

THE UNUSUAL EXPEDIENT of adding two new stories of steel framing to an old three-story timber building (and also reframing the old third story in steel) was adopted by Eli Lilly & Co., pharmaceutical manufacturers, in enlarging a stock warehouse at its Indianapolis plant. The two new stories provide expanded facilities for the office forces and are air conditioned, requiring double wall construction to provide space for air ducts. Operations in the stock house in the lower stories could not be interrupted during the alteration, and this affected the construction plan to the extent that the new fourth floor was built inside the building before the old roof was removed, so that the new floor could serve as a roof during the construction. The most novel single element in the reconstruction is the splice between the old timber columns and the new steel columns.

The decision to use steel for the added stories was based upon a study which indicated that the remaining useful life in the timber structure could be equated




**SPLICE DETAIL** between timber and steel column. Original timber girders and beams were retained on this third floor. Two new upper floors utilize a cellular steel plate design with concrete slab surfacing.



to the economic life of the building as a unit of the plant. Therefore, the choice of material for the new addition rested first on its adaptability to rapid construction (in order not to interrupt manufacturing operations unduly), followed by low cost and high salvage value. The new construction also needed to be as light as possible since the old building had not been designed for additional stories. Cellular steel plate floors, accordingly, were specified along with the steel framing while light-weight concrete blocks were used for back-up in the new exterior wall construction.

The old building about 70 ft. (4 bays) wide and 175 ft. (9 bays) long was of bearing-wall construction. Its interior framing consists of square timber columns, 15½ in. in the lower stories and 13½ in. in the upper stories, flitch plate girders composed of two 7½x17½-in. timbers placed either side of a 1⅜-in. steel web plate, and timber beams. An ingenious column splice placed below each floor serves also as a support for the girders. The splice consists of an 18-in. I-beam, 3 ft. long, laid flat over the top of the column with the upstanding flanges forming a channel into which the girders fit; the upper column also rests in this channel between the ends of adjacent girders. Built about 25 years ago of long-leaf yellow pine, the old frame was found to be in excellent condition.

In the reconstructed building, bearing-wall construction is retained at the sides and at the back end; in the front end steel columns were installed from the footings to the roof, as an aid in making the new street facade correspond to that on an adjacent office building. On the interior, the wood columns in the third story (about 30 in number) were replaced with steel columns on top of which the two new stories of steelwork were erected.

All new steel is in wide-flanged sections—columns and beams 8 in. deep and girders 14 in. deep. The new floors are of cellular steel plate type, 3¼ in. deep, on top of which is a 2¾-in. concrete floor. The dead load from this construction is only 50 lb. per sq.ft. made up of 33 lb. for the concrete, 8 lb. for the steel floor, and 9 lb. for metal lath and plaster in a suspended ceiling. A live load of 100 lb. per sq.ft. is used in the design of the new framework. The new roof is of timber construction.

As previously stated, the column splices between the old timber and new steel columns just below the third floor level represent a novel detail. This is so because the old timber column splice is utilized without alteration except for the addition of a few brace plates. In effect, the top story length of the old timber column was lifted out and the new steel column inserted in its place between the ends of the flitch plate girders and resting on the web of the 18-in. I-beam column cap. As shown in the accompanying sketch, the bracing consists of ⅜-in. plates, extending down along the sides of the wood column and up over the edges of the steel column caps where they are bent in to a connection with the web of the new steel column. The sides of the wood column are enclosed in short lengths of 18-in. channels against which the plates bear under pressure of two ¾-in. bolts extending through the column. The old timber floorbeams are attached to the new steel column by 8x6x⅜-in. clip angles. The balance of the steel framing is of conventional type and design.

The reconstructed building was designed by Robert Frost Daggett, architect of Indianapolis. The original timber building was designed by Mr. Daggert's father. William A. Hanley is director of engineering of Eli Lilly & Co.

# *Letters to the Editor*

## Analyzing Distribution Networks

Sir—The application of the Hardy Cross method to the solution of hydraulic problems relating to water distribution systems, so clearly described by Prof. Doland in his article "Simplified Analysis of Flow in Water Distribution Systems" (*ENR*, Oct. 1, 1936, p. 475), is an important step forward. Profs. Cross and Doland are to be congratulated for bringing this excellent method to the attention of the waterworks profession.

This is not the first workable method for solving pipe network problems as intimated in your editorial. The method employing an electric network analyzer described by the writer and Prof. H. L. Hazen (Jour. N.E.W.W.A., Dec. 1934) and referred to by Prof. Doland is equally workable. In the Cross-Doland method, however, no expensive electrical equipment is required as is the case with the Camp-Hazen method. The new method may therefore be used by anyone who possesses a good slide rule and a sharp pencil.

The problem solved by Prof. Doland in illustrating the use of the method is a modification of the problem solved by the writer and Prof. Hazen using an electric network analyzer. No direct comparison can be made between the speed of the two methods on the basis of Doland's solution, which, he stated, required about 4 hr. to make.

In order to obtain a comparison between the two methods and a better understanding of the Cross - Doland method, the writer has repeated the solution of the problem previously solved with the electric network analyzer by means of the new method. Three corrections were required in the new method in order to obtain results comparable in accuracy to those obtained on the electric network analyzer with three adjustments of the resistors. The time required by means of the new method was about 6 hr. The time required for three balancing operations on the analyzer was about four hours, with two men working. In addition about ten man-hours was required in selecting resistors and connecting up the analyzer board. The new method is therefore more rapid than the electric analyzer method, provided no more than three corrections are required for solution. The writer wonders if more complicated networks than the one used in the illustration might not require a larger number of corrections in order to secure sufficient accuracy.

Cambridge, Mass.,
Oct. 7, 1936

THOMAS R. CAMP
Associate Professor of Sanitary Engineering
Massachusetts Institute of Technology

## Weld Metal Improves

Sir—In your editorial "Progress in Steel Design," (*ENR*, Sept. 10, 1936, p. 385) it is mentioned that "the American Institute of Steel Construction provides for a basic unit stress of 20,000 lb. per sq.in." Further on in the paragraph it says, "The advance to 20,000 lb. is thus a natural consequence of better steel manufacture." It might be interesting to note in this connection that in the older days of welding the maximum ultimate tensile strength of weld metal was on the order of 45,000 to 50,000 lb. per sq.in. while at the present time there are electrodes on the market giving 115,000 lb. per sq.in. This advance in the physical qualities of weld metal has been made in a relatively few years and permits the use of stresses very much higher than were previously possible. In addition to the increase in ultimate strength, there has been a marked increase in the yield point, and without sacrificing ductility.

This high quality weld metal permits the use of increased working stresses but at the same time it requiries that the designer know a great deal more about the details of his design, because the designing is done much more accurately than was the case some time ago.

Cleveland, Ohio,
Sept. 29, 1936.

E. W. P. SMITH
Consulting Engineer,
Lincoln Electric Co.

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Editorial Offices at 330 West 42d Street, New York

November 12, 1936


## *Aiding the Hingeless Arch*

HINGLESS STEEL ARCHES are rare enough to arouse interest. When, as in the Henry Hudson Bridge at New York, they attain such record span lengths as 800 ft. and moreover are of rib type instead of the more usual spandrel-braced design, they invite special consideration. Boasting such advantages as maximum rigidity, minimum bending moments and economy of material, fixed arches nevertheless are seldom built, while the less efficient hinged design is common. The fixed arch is a victim of increased knowledge —knowledge of the necessity for controlling the position of the pressure line and of the extreme accuracy imposed upon fabrication and erection in securing this control in a hingeless design. Many old masonry arches, built without much concern for ultimate deflections, have stood for centuries only because they developed virtual hinges at some weak point which allowed the pressure line to adjust itself to a safe position. Also, the lack of precision in fabrication and erection of early hingeless steel arches suggest that the designer's desires may not have been wholly met. But the fabrication and erection of the Henry Hudson Bridge gives proof of the scientific fidelity of which the present-day construction art is capable. The apparent ease with which the results were attained suggests that some traditional objections to fixed arches no longer obtain. By demonstrating that practically perfect fabrication as well as reasonably accurate stress imposition are possible, the Henry Hudson Bridge has advanced the cause of the hingeless steel arch.

## *Rural Freeways*

CONCEPTION OF THE MAIN ROAD as a traffic freeway has developed rapidly. The discussion of M. W. Torkelson that we published three weeks ago, followed in this issue by F. C. Lawton's bold argument for intercity arterial roads and E. S. Draper's description of the Tennessee Valley freeway, gives evidence of the new thinking. Throughout the country the leading highway engineers are working for wider rights of way and for a degree of roadside control that points to increased freedom of traffic. This freedom, incidentally, can be attained even where the land is not purchased outright, as for the TVA freeway, but is taken under easement only. It has been found that most owners are willing to accept reasonable limitation of access and of roadside use whenever the highway officials urge the matter with adequate conviction. But not all highway engineers are yet sufficiently conscious of the situation. Though they may appreciate traffic freedom in the abstract, many still lack the practical realization that road service today demands it as commandingly as it ever demanded smooth and stable travel surfaces, and that this freedom can be obtained through farmland and forest as fully as through those congested metropolitan areas that can afford superhighways with continuous marginal service roads. The rural freeway problem is so much simpler than that of the metropolitan superhighway that the engineer who does not keep it in view in his road design and construction planning is overlooking his largest opportunity.

## *Purification Methods Extended*

SELF-PURIFICATION OF RIVERS by reaeration and associated processes is one of the most potent forces at the command of the sanitary engineer. For many years he has known that when the entrance of wastes is controlled within certain limits a stream can perform a very large amount of purification without harmful effects. Now, by extending the application of this knowledge to lakes and ponds, the scope of purification methods has been definitely enlarged. The success achieved in the Texas work described in this issue deserves close study, as a suggestion of how far the principle may be adapted to the requirements of other cases. Here an artificial lake provides a means of carrying on effective secondary sewage treatment at low cost. The predecessor of the Texas installation, the impounded reservoirs on the Ruhr River in Germany, owed its origin to compelling necessity, which with Imhoff's insight into the nature of purification processes led to the creation of a new form of treatment. These reservoirs serve to store sewage-polluted water long enough for effective stabilization of the organic matter by reaeration and bacterial action, and incidentally serve also for flood control and power generation. Under quite different conditions, the Texas installation gives fuller proof of the value of ponding treatment. Operating results show a minimum absorption of 50 lb. of oxygen per acre per day, and since this value is obtained by absorption alone, without addition of oxygen by dilution water, it represents a dependable measure of available pond oxygenation. The noteworthy detail features of the Texas plant, including bottom distribution and final spray aeration, are suggestions of the further engineering development that the principle may receive. It becomes apparent that secondary pond treatment following sedimentation has prospects of wide use.

## *Judgment and Stresses*

IT IS A GRATIFYING SIGN of advance in the art when well crystallized engineering rules like those fixing working stresses are reviewed and raised to higher level with the general approval of the profession. We are thinking of the recent raise in basic working stress for structural steel from 18,000 to 20,000, which has met with nothing but favorable discussion. Such action usually expresses a pre-existing but inarticulate general conclusion from experience that the change is desirable. Undoubtedly this is the case with the steel stress increase, just as it was the case with the step-by-step advance in concrete stress that has been under way since concrete practice first grew to adult proportions and that is still going on —for if we do not misjudge, concrete stresses may experience further increase. But it is well to remember that the use of such increased standards must be accompanied by a correspondingly increased application of engineering judgment. Low working stresses can stand more carelessness and abuse than high working stresses; the latter are sensitive to many factors of omis-

sion or commission that lie in the engineer's forethought. The shovel-and-bucket concrete of 450-lb. days will not do in times of 800 to 1,200-lb. practice; the steel that at 16,000 would not rebel against handbook design and brute-force construction is distinctly less tolerant under a 20,000-lb. impost. Engineering judgment, we submit, must be an integral part of working-stress practice if our modern high stresses are to prove satisfactory. The greater competence of the art that is reflected in increased working stresses cannot be written into building codes and standard specifications; but disappointing results are likely if it is not there in fact.

## Out of a Store of Knowledge

THE EYES OF THE NATION are focused this week on an engineering achievement. As traffic is admitted to the upper deck of the Bay Ridge at San Francisco, newspaper, radio and newsreel will tell of the technical skill that made the event possible and emphasize the importance of a physical tie between San Francisco and its neighbor cities across the Bay. Tomorrow it will be just another bridge.

San Francisco Bay has challenged bridge builders for generations. The superlative difficulties that made the proposals of earlier years seem fantastic finally were outweighed by the imperative need for a transbay roadway, and then even the greatest engineering difficulties melted into routine. Backsight on the achievement reveals nothing more striking than the unerring way in which existing knowledge was fashioned by efficient engineering and ingenious construction processes into the world's greatest bridging enterprise. From that day in 1930 when the borings of the Hoover-Young Commission had established the presence of a ridge of rock between Rincon Hill and Yerba Buena Island, each step was so logically based on engineering precedent that the project seemingly moved ahead under its own momentum. That it did not so move is of course known to all those who have followed its successive achievements.

The greatest of these center in the foundations. The twin suspension bridges invoked no essentially new principles; the 1,400-ft. cantilever, longest in the United States, is unique only in some special details. But the foundations push back the borders of the impossible by going to depths never before reached, in water deeper than ever before encountered. The multiple-dome method of controlling caisson buoyancy in the west bay crossing provides a new tool that may make even greater performance possible; the center anchorage monolith sunk by this method, extending 220 ft. below water and rising 282 ft. into the air, claims a place alongside the world's greatest engineering works. In the east bay crossing, the 240-ft. depth reached by one of the piers is hardly less notable an achievement. And in the superstructure the tower erection by hammerhead derricks, four-wire cable spinning, copper-bearing steel used in large tonnages, enrich the records of engineering accomplishment.

Solution of the formidable problems of the Bay Bridge as though they were no more than routine testifies to the genius and skill enlisted in the execution of the project. But when all things are considered, one comes back to the fact that the bridge is a product of what existing stores of knowledge made available. It typifies the service of modern engineering to the world.

## Construction Looks Ahead

FOR A LONG TIME construction men as well as others have found the future too uncertain to look beyond immediate conditions. The election returns clear away uncertainties and open the way to appraisal of the future. The heavy vote and the large majority for the administration assure continuity of national policy; except as the ending of the economic emergency calls for change, the next year at least is certain to follow the course of the last two or three. What does this mean for the interests of engineering and construction?

The general factors are favorable. Recovery, industrial resumption and expansion are bound to progress. Construction therefore should maintain and accelerate its two-year rise, and in putting to work more of its eight million workers it would bring about an increase in purchasing power and the pace of business that would encourage further construction.

Private construction enterprise, which already is at a substantial level, cannot fail to make further advances. Housebuilding growth is likely to call another half-million men to employment next year. In the field of public works flood-control operations will be large, under the pressure of the disasters of last spring, but otherwise we have probably seen the last of the big federal projects unless the St. Lawrence enthusiasts gain their point. Reclamation, though assured of steady support, has not made out a case for new projects. Highways should continue unchanged; federal housing appears over; soil conservation operations are uncertain but have little meaning for construction. Meantime municipal improvement needs are becoming imperative, though unfortunately the return of public-works initiative to states and cities is seriously retarded.

In sum total, maintained public construction and rapidly growing private enterprise spell hope for a substantial rise of construction from the depression slough in which it still continues.

Against this, important unfavorable conditions give cause for question. Chief among them is the harm done by the WPA's trespass on normal construction and maintenance, which throttles the growth of working opportunities, paralyzes construction revival and delays the restoration of local responsibility. Survival of the PWA system, no longer needed, is another retarding factor. And finally, dictation of improvements by executive fiat under blank-check appropriation casts confusion into both private and public planning; for, regardless of electoral majorities, it is quite certain that the country wants no more Quoddies.

Problems of another kind lie in the shortages of skilled workers that are already becoming apparent. Evidently, re-training on a large scale is necessary, but labor is not interested in the problem, and government and the industry sidestep it. Building up an efficient, adequate and satisfied working force is in fact becoming an imperative need, for no single feature of the election was more significant than the emphasis that it laid on the worker's claim to consideration.

These complicating problems deserve the earnest study of all construction interests. In addition, construction has need to put and keep its house in order, so that it may merit the public's respect and confidence in all its practices. The spirit of the times calls upon every industry and vocation for an account of its conduct and demands that it make the public interest its standard.

# CURRENT NEWS

## Construction and The New Congress

**A summary indicating the outlook for federal action on construction in the coming legislative session**

***Washington Correspondence***

WPA PROGRAM: Fundamental changes in the WPA organization can be predicted confidently, but WPA will have to continue about as it is now organized through this winter, because alternate plans for meeting unemployment require further legislation. Such alternate plans will probably include a vigorous effort to secure ratification of the child-labor amendment and legislation for federal aid to keep youth in school longer. However, WPA will probably have to have at least $500,000,000 in additional money to carry through the winter, since present funds cannot last longer than Feb. 1.

PWA PROGRAM: PWA has allotted $104,000,000 since July 28, but very little of this has come out of the $300,000,000 earmarked at the last Congress. The existence of this large fund for non-federal projects will probably militate against securing any additional money at the next session of Congress. However, Congress will probably appropriate money for construction of federal buildings. Prospects for PWA's housing program are discussed below.

HOUSING: That section of the FHA providing for modernization loans expires on April 1. No effort will be made to extend it. The section covering mutual insurance on mortgages continues indefinitely, except for the provision of government guarantee of principal and interest on debentures issued to cover defaulted mortgages, which expires July 1. Extension of this last will be sought, but will be fought by building and loan associations. However, Congress will probably extend this provision.

PWA's non-federal housing program is virtually stopped by the terms of the 1936 appropriation bill which excluded such activities. A few projects may go through if rescissions from older funds makes the money available. Also, PWA will absorb the suburban housing activities of the Resettlement Administration; however, it will only complete work which is already started. With two-thirds of its last appropriation still unallocated, PWA has no reason for requesting further funds.

The Public Housing Authority plan will be revived and a revised form of the Wagner Bill will probably be passed. The details are still uncertain, but the result should be several hundred million in low-rent housing construction within the next decade.

Many schemes are current for helping the average citizen to finance home construction. Some one of these might result in passage of a law if interested parties could get together on a workable plan. However, this is not likely to occur this session, and better times will reduce the pressure for it.

FLOOD CONTROL: Congress is expected to appropriate at least $100,000,000 for flood control work authorized under the terms of the Omnibus Act passed at the last session as a result of the 1936 flood.

RECLAMATION: The Reclamation Bureau looks forward to a continuation of the normal program, which now involves annual expenditures of fifty to sixty million. The most important decision before

(*Continued on page 698*)

## Bridge Collapses After Collision

**Narrow bridge at Red Bluff, California, falls after truss member is struck by a loaded truck**

A 150-ft. span of a 5-span steel truss bridge over the Sacramento River on U.S. 99E at Red Bluff, Calif., collapsed on the morning of Oct. 27 after it had been struck by a truck crossing it. The collapse dropped into the water 30 ft. below a truck and trailer, each loaded with 5,000 gal. of gasoline, and a second truck loaded with empty oil drums. The drivers escaped with only minor injuries.

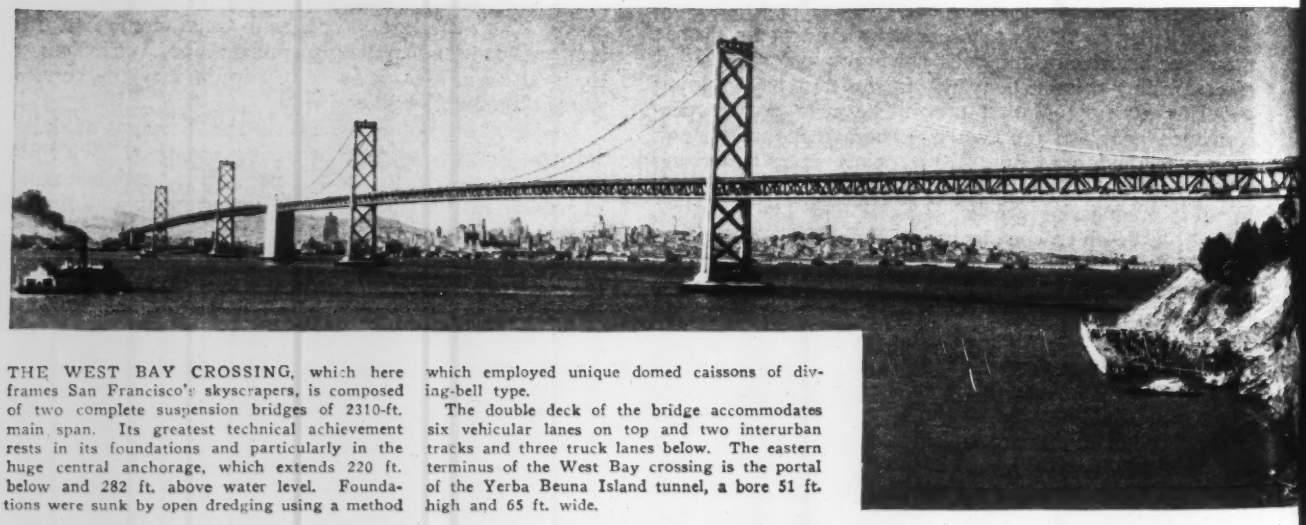
Inspection of the wreckage indicated that the gas truck had crashed through the timber railing, and had struck a vertical post of the truss, the bending of which caused the collapse.

The bridge consists of five 150-ft. steel Pratt trusses and one 48-ft. pony truss. The roadway width is only 16½ ft. between curbs. The bridge is reported to have been built by the Tehama Co. in 1884 and was remodeled in 1919.

The fallen span was the most easterly of the 150-ft. trusses, and was next to the pony truss. As the span fell it tipped forward the top of the high cylindrical pier at its east end, leaving the adjacent 40 ft. pony truss only partially supported at the southwest corner. Plans were immediately made by the state highway department for closing the gap temporarily with a timber trestle. However, on the following day, while the pony truss was being removed, it fell to the ground, killing one worker.

It is expected that the bridge will be replaced by the state with a permanent structure in 1937.

**VEHICULAR TRAFFIC CLAIMS UPPER DECK OF SAN FRANCISCO-OAKLAND BRIDGE TODAY, NOVEMBER 12**



**THE WEST BAY CROSSING, which here frames San Francisco's skyscrapers, is composed of two complete suspension bridges of 2310-ft. main span. Its greatest technical achievement rests in its foundations and particularly in the huge central anchorage, which extends 220 ft. below and 282 ft. above water level. Foundations were sunk by open dredging using a method which employed unique domed caissons of diving-bell type.**

**The double deck of the bridge accommodates six vehicular lanes on top and two interurban tracks and three truck lanes below. The eastern terminus of the West Bay crossing is the portal of the Yerba Beuna Island tunnel, a bore 51 ft. high and 65 ft. wide.**

# Transport Department Formed in Canada

**New federal department merges railways and canals department, marine department, and civil aviation**

The new Canadian Department of Transport which takes over the duties of the former Railways & Canals Department, the Marine Department, and the civil aviation branch of the National Defense Department, was organized on October 2, in Ottawa.

C. E. Howe is minister of the new department. C. I. Smart has been appointed deputy minister; and George W. Yates, assistant deputy minister. R. K. Smith will be director of marine services, Com. C. T. Edwards, chief of air services, and H. F. Alward, departmental counsel.

Col. Smart, now deputy minister of the department, was formerly deputy minister of the Railways & Canals Department. R. K. Smith, the new director of marine services, was deputy minister of marine in the former set-up. As director of the marine services he will now have charge of aids to navigation, steamship inspection, nautical division, agencies and supplies, and pilotage. Marine engineering and supervision of public harbors will be under the direction of J. G. MacPhail, formerly in charge of the commission of lights branch.

## Argentina Asks for Bids on Big Lift Bridge

Bids will be received at Buenos Aires by the Direction National de Vialadid on Dec. 7, 1936 on the concrete foundation, piers, and approaches for the new high-level lift bridge over the Riachuelo River at Buenos Aires.

The bridge will have a 40-ft. roadway and two 8-ft. sidewalks; the lift span will be 200-ft. long. The normal level of the bridge will be 80-ft. above the water, requiring approaches nearly 2,000 ft. long on each side of the river. The lift span will rise to a height of 150 ft. over the water for the passage of large vessels. Two mechanical escalators are being provided for pedestrians at each of the two piers with a capacity of 8,000 passengers per hour.

The present contract is estimated to cost about $2,200,000, including the purchase of land and rights-of-way. The 200-ft. lift span will be contracted for separately.

## Nashville Court Upholds Joint Suit Against TVA

A new effort by government attorneys to halt the suit brought in Nashville district courts against TVA by nineteen Southern power companies failed when Judge Gore overruled a motion that the suit be dismissed on the grounds that the companies did not have any joint action but should bring separate actions against the TVA. Government attorneys claimed that some of the nineteen power companies have not been affected by the Authority's hydro-electric development on the Tennessee River and that those companies which have been affected have been affected differently and should file separate suits.

Judge Gore held, however, that although the utilities may be interested in the outcome of the case in varying degree, all are seeking the same relief, a court order holding that some or all of the TVA power activities are either unconstitutional or in excess of the TVA act. Judge Gore held that the utilities are correct in their contention that the directors of the TVA are "engaged in carrying out a single program, plan, or conspiracy, the execution of which will irreparably injure the property and business of each of the complainants."

Counsel for the power companies have asked a temporary injunction to halt any expansion of the TVA power program pending a final decision in the suit. No date for a hearing on this injunction has been set but it is expected that it will be argued soon.

# Full-Time Official For E.C.P.D.

**R. I. Rees Becomes Full-Time Vice-Chairman—Carnegie Foundation Gives $16,000 for expenses**

The Engineers Council for Professional Development has announced that R. I. Rees, vice-chairman of the Council, will devote his full time to the duties of that office. Mr. Rees is chairman of the committee on professional training and is chairman of the committee on ways and means of the Engineers Council for Professional Development.

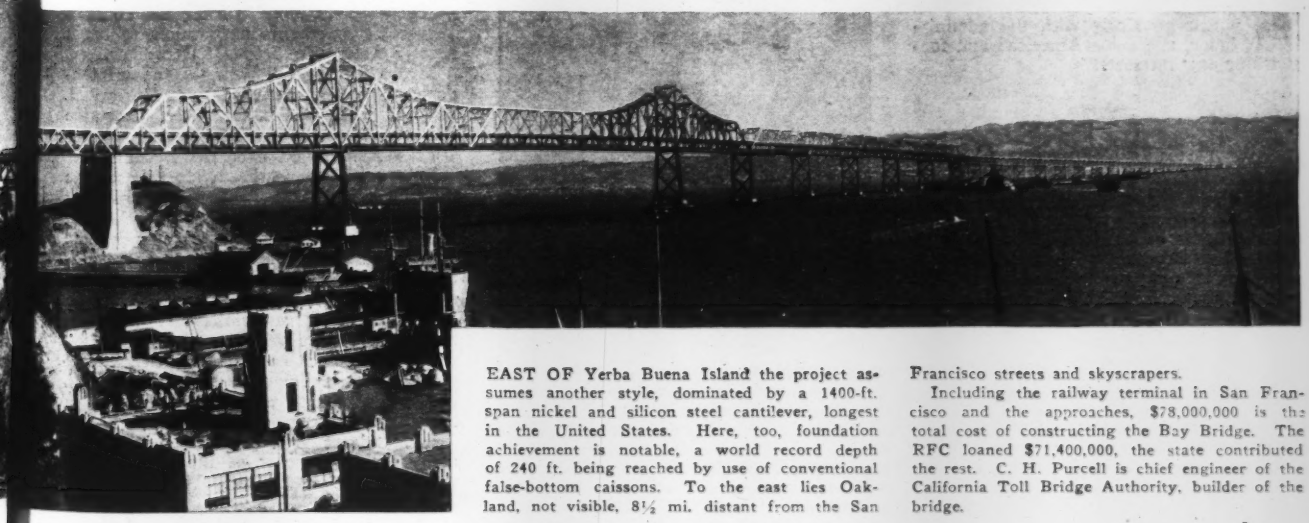
At the same time it was announced that the Carnegie Foundation for the Advancement of Teaching had granted $16,000 to the Engineers Council for Professional Development, to meet the cost of its work for the year of Oct. 1, 1936, to Sept. 30, 1937. Other funds for the support of the Engineers Council have been provided, in the amount of $3,450, by the Engineering Foundation. The rental for the recently opened office of the Council in the Engineering Societies Building is expected to be met by three of the participating societies, the American Society of Mechanical Engineers, the American Society of Civil Engineers, and the American Institute of Electrical Engineers.

## Buffalo Engineers Form Central Organization

The Engineering Foundation of Buffalo has recently been organized to further the interests of the engineering profession in the Buffalo, N. Y. district. The Foundation proposes to sponsor several large meetings of interest to the profession in general.

John G. Ullman has been appointed chairman of the newly formed organization. Mr. Ullman has long been actively connected with the Engineering Society of Buffalo.

**FORTY MONTHS AFTER CONSTRUCTION BEGAN ON THIS GREATEST OF BRIDGE BUILDING PROJECTS.**



**EAST OF Yerba Buena Island the project assumes another style, dominated by a 1400-ft. span nickel and silicon steel cantilever, longest in the United States. Here, too, foundation achievement is notable, a world record depth of 240 ft. being reached by use of conventional false-bottom caissons. To the east lies Oakland, not visible, 8½ mi. distant from the San Francisco streets and skyscrapers.**

**Including the railway terminal in San Francisco and the approaches, $78,000,000 is the total cost of constructing the Bay Bridge. The RFC loaned $71,400,000, the state contributed the rest. C. H. Purcell is chief engineer of the California Toll Bridge Authority, builder of the bridge.**

# The New Congress

(*Continued from page 696*)

the next Congress is on the continuation of work at Grand Coulee. Out of a total estimated cost of $186,000,000, $63,000,000 has been provided so far and is entirely obligated. Other projects on which construction has started but which need more funds for completion include Yakima Roza, Payette, Heart Mountain, and Gila. The Reclamation Bureau points to the continued demand for irrigated land, which has been intensified by drought experience, as basis for the belief that Congress will look kindly on further appropriations.

The Columbia Basin project is still in suspense. The Bureau feels that further continuation of Grand Coulee is the first objective and that it is too soon to start work on irrigation projects until the necessary power supply is assured. President Roosevelt's known favorable attitude toward Grand Coulee indicates that this project will probably continue without interruption.

Definite figures on what requests the Bureau will make for additional funds cannot be had until the next budget is released.

ST. LAWRENCE SEAWAY: Proponents of the St. Lawrence seaway project claim 50 Senatorial yeas, 24 nays, and 22 doubtful. Fourteen of the doubtful Senators will be needed to ratify the Canadian treaty at the next session. Opposition which has formerly existed in the far Southeast will be disposed of by a trade for Florida ship canal support.

Although he did not mention it in his campaign, Roosevelt wrote on Sept. 16, "It is my very earnest hope that negotiations for the St. Lawrence project may be carried forward within the coming year to a successful and final conclusion." The general opinion is that he will urge ratification of the treaty.

Against the St. Lawrence seaway will stand some strong railroads, some strong power interests, a nearly solid line-up of the Atlantic seaport states and Louisiana, and the fact that the United States no longer has quantities of wheat for export, which was the original argument for opening the St. Lawrence to deep draft shipping. In favor of the project will be the lake and midwest states, all of the disinterested states, which go along with the administration, labor, and some American and foreign shipping interests.

## New York Is Preparing Code Interpreting Silicosis Law

An industrial code for the administration and operation of the New York occupational disease compensation act, insofar as it applies to rock drilling and excavation, is in the course of preparation by the state department of labor and industry. The act, passed by the state legislature last spring and in effect since Sept. 1, provides for the compensation of silicosis as an occupational disease and requires the use of dust control methods on all rock work within the state. A committee of contractors, mine operators, insurance representatives, medical and health officials, and rock drill and dust control device manufacturers, appointed by industrial commissioner Elmer F. Andrews, has been at work since July in compiling a workable code of administration satisfactory to all. The code, now completed and awaiting the approval of the industrial board, defines safe hygienic limits of dust concentration, outlines standard procedure for testing of dust control equipment and methods, and lists certain rules and regulations for the law's operation. Public hearings will be held, prior to final adoption of the code, by the industrial board, after which it becomes part of the law.

In the meantime testing of rock drill dust control apparatus has been started on an outcrop of hard granite on a parcel of state property in Rockland County. Every type of control method must be tested and approved by the state before it can be used. Tests are being conducted by Theodore Hatch, associate dust control engineer, division of industrial hygiene. A description of the test procedure will be published in an early issue.

Pending approval of control methods and apparatus, enforcement of the law is at a standstill.

**WRECKAGE** left after collapse of a 150-ft. truss over the Sacramento River at Red Bluff, Calif. Story on page 696.

*Sacramento Bee Photo.*

# *Washington Highlights*

**by Paul Wooton**

Amendments to the federal aid law increasing the amount of federal aid for secondary roads will be presented to the next session of Congress. At the last session, $25,000,000 was appropriated for secondary roads for each of the two fiscal years. An effort will be made to increase this to an annual contribution of $50,000,000.

Regardless of the outcome of its negotiations for a permanent peace with the Commonwealth and Southern System, TVA is expecting great increases in the Tennessee Valley power consumption. The original plan to defer for an indefinite period the installation of generating equipment at Pickwick Landing Dam has been changed, and TVA now intends to make an initial installation of two units with a total capacity of 90,000 hp. This will bring the total installed capacity in dams now under construction to 573,400 hp. out of an ultimate capacity of 1,858,000 horsepower.

Snow removal, it has been decided, will not be undertaken this winter as a part of the WPA program. The WPA has decided that snow removal must be regarded as a normal function of local government units. State officials of the WPA have been instructed not to submit applications contemplating the removal of snow from roads, streets and highways. However, where a shortage of labor for snow removal exists, WPA workers will be made available to the municipalities and will be permitted to return to their WPA jobs upon completion of the snow removal work. Instructions have also been issued that if there is a shortage of trucks for snow removal work, WPA trucks will be temporarily released, but the public agencies charged with snow removal will be required to pay rental on the trucks.

## C. K. Calvert Joins Staff of Indianapolis Water Company

Cevil K. Calvert has submitted his resignation as superintendent of the Indianapolis sewage disposal plant, to take effect Nov. 30, and has accepted an appointment as chemical engineer for the Indianapolis Water Co., with his new duties to begin shortly after the first of December.

Mr. Calvert will fill the superintendency recently resigned by Harry D. Jordan, who will assume the executive secretaryship of the American Water Works Association early in December.

Before his connection with the sewage disposal plant, Mr. Calvert was assistant chief chemist and bacteriologist for the Indianapolis Water Co., with immediate supervision over operations of the water treatment and filter plant. He joined the staff of the sewage disposal plant in 1921, becoming superintendent in 1925. During the past year he has supervised the design and construction of several plant additions for the sewer department, the last of which was recently placed in operation.

Mr. Calvert is a past-president and is at present a director of the Indiana section of the American Water Works Association.

## Flood Control Bond Issue Defeated in Washington

At the recent general election a referendum bill was defeated which would have authorized the state of Washington to incur an indebtedness of $15,000,000 to match federal and local funds for flood control work (*ENR*, Oct. 22, 1936, p. 594).

At the same election an amendment to the state constitution which would have authorized the state to engage in the production and wholesale distribution of electric energy and to assist in the development of hydro-electric generating plants was also defeated. This measure was intended to enable the state to market power to be produced at Grand Coulee Dam.

A similar bill for the marketing of electric power in Oregon was also defeated.

## SOCIETY CALENDAR

INTERNATIONAL ACETYLENE ASSOCIATION, annual convention, St. Louis, Mo. Nov. 18-20.

ASSOCIATED GENERAL CONTRACTORS, fall meeting of governing board, Washington, D. C., November 16-17.

HIGHWAY RESEARCH BOARD of the NATIONAL RESEARCH COUNCIL, annual meeting, Washington, D. C., Nov. 18-20.

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS, annual meeting, San Francisco, Calif., December 7-10.

AMERICAN ROAD BUILDERS ASSOCIATION, convention and highway exhibit, New Orleans, La., Jan. 11-15.

AMERICAN ENGINEERING COUNCIL, annual assembly, Washington, D. C., Jan. 14-16.

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NEW JERSEY STATE LEAGUE OF MUNICIPALITIES, annual convention, Trenton, N. J., Nov. 19-20.

NEW JERSEY HEALTH AND SANITARY ASSOCIATION, annual meeting, Atlantic City, N. J., Nov. 20-21.

ASSOCIATION OF WESTERN STATE ENGINEERS, annual meeting, Santa Fe, N. M., Dec. 3-5.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION, MIDDLE ATLANTIC SECTION, fall meeting, New York City, Dec. 5.

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IOWA, examination for professional engineering on Dec. 1-2, and for land surveying on Dec. 1-2, and for land surveying on Dec. 3, in the Senate Chamber at the State House, Des Moines, Iowa.

SOUTH CAROLINA, examination for professional engineers and land surveyors on Dec. 1, in Columbia in the office of the Board at 1218 Senate St. Applications must be filed with the Secretary of State Board of Engineering Examiners before Nov. 21.

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At a meeting of the NEW YORK SECTION of the AMERICAN WATER WORKS ASSOCIATION held at Saratoga Springs last month it was decided that not spring meeting of the New York section would be held in 1937, due to the fact that it would conflict with the annual meeting of the American Water Works Association.

The October 26 meeting of the MISSOURI VALLEY SECTION of the AMERICAN WATER WORKS ASSOCIATION elected the following officers: chairman, N. T. Veatch, Jr., Kansas City, Mo.; vice-chairman, Dale Maffitt, Des Moines, Iowa. The following directors were elected: South Dakota, W. W. Towne, Pierre; Kansas, Bernard Ulrich, Manhattan; Iowa, Walter Molis, Muscatine; Nebraska, Rex Henry, Fremont; Missouri, W. Victor Weir, St. Louis.

### NEW FORTH BRIDGE A 364-FT. SWING SPAN

FORMALLY opened to traffic on October 29, a new bridge over the River Forth, at Kincardine, Scotland, provides a needed connection between the mining areas of the north and Glasgow, and the southwest industrial district. It is situated about 13 miles above the famous Forth cantilever bridge and contains a swing span of 364 ft., the longest in the United Kingdom, as well as fourteen 100-ft. steel girder spans and about 700 ft. of concrete viaduct. The deck accommodates a 20-ft. roadway and two 5-ft. sidewalks. The center pier of the swing span is composed of a circle of six 14½-ft. diameter cylinders sunk to rock under air and supporting a hollow cylinder 42 ft. in diameter with walls 5 ft. thick. The consulting engineers are Sir Alexander Gibb & Partners.

## Louisiana Approves Highway Bonds

Constitutional amendments approved by Louisiana voters in the general election Nov. 3 will provide $35,000,000 to the Highway Commission and $20,000,000 to the city of New Orleans for street paving and municipal improvements.

A bond issue of $30,000,000, based on gasoline tax receipts, was approved. Of this, $5,000,000 will be used for a new highway-railway bridge over the Mississippi River at Baton Rouge; $3,000,000 will be used for secondary road construction; $6,500,000 for retirement of indebtedness; and the remaining $15,500,000 for completion of paving of major highways and for purchase of new equipment.

A further bond issue of $5,000,000, guaranteed by receipts from vehicle taxes in parishes near Lake Ponchartrain will be used for purchase of the Ponchartrain Bridge Co. toll bridge across the narrow end of the lake and for the completion of subgrade paving of a route connecting New Orleans via the border of Lake Ponchartrain to the Hammond, La.-Jackson, Miss. highway at Freniet.

At the same election the city of New Orleans obtained permission to issue $20,000,000 in bonds to provide funds for street paving and other municipal improvements. The issue will be financed from revenues derived from taxes on gasoline.

## Resistance Continues to Walsh-Healey Act

Bids received on the re-advertisement of two contracts for 28 motor trucks to be used by the Resettlement Administration indicate continued resistance to the Walsh-Healey Act by the automobile industry. The original bids were thrown out, as only one bidder, the A. J. Corbitt Co., indicated its willingness to accept the Walsh-Healey restrictions by submitting a full bid (*ENR*, Oct. 29, 1936, p. 627). The other bidders all placed their bids under the $10,000 exception by making partial bids.

On the second opening, on Nov. 2, the Corbitt Co. repeated its bid, but all the others refrained from bidding with the exception of Reo, which offered to supply only six trucks. Although a tendency to bid through dealers had been expected, this failed to develop.

Since the Walsh-Healey government contract act, specifying minimum working standards for manufacturers or dealers furnishing materials, supplies and equipment to the government, went into effect, 58 contracts have been awarded in which the bidders agreed to abide by the Walsh-Healey stipulations, according to an announcement made on Nov. 7 by the Department of Labor. The great majority of these awards have gone to small manufacturers or local distributing organizations, but several large industrial units have bid.

## Obituary

CONRAD TRIMBORN, president and treasurer of the Milwaukee Bridge Co., died on Oct. 30 in Milwaukee at the age of 70.

PROF. FREDERICK R. FAULKNER, head of the civil engineering department of the Nova Scotia Technical College (affiliated with Dalhousie University) died in Halifax on November 4 at the age of 58. Professor Faulkner was a graduate of Acadia University and the Massachusetts Institute of Technology.

SIR NEWTON JAMES MOORE, president of the Dominion Steel & Coal Corporation and formerly premier of Western Australia, died recently in London, England. Sir Newton was for many years active in Australian and English politics. He was for several years vice-president of the Dominion Steel Corporation in charge of European operations. He was the first president of the Institute of Surveyors of Western Australia.

BERTRAND E. GRANT, division engineer, Chicago Board of Local Improvements, died on Oct. 30 at the age of 68. Mr. Grant was a graduate of Rensselaer Polytechnic Institute in 1890. From his graduation until 1898, Mr. Grant served on the engineering staff of the Chicago Drainage Canal, except for one year as assistant city engineer in Waterbury, Conn. He practised as a consulting structural engineer from 1890 to 1905. In the latter year he entered the service of the Chicago Local Improvement Board, with which he has been connected ever since. In 1917 Mr. Grant was president of the Western Society of Engineers.

ANTHONY TOOHEY, a supervisor of steel construction for the Bethlehem Steel Co., died in Hamilton, Ohio, on Oct. 28 at the age of 70. Mr. Toohey had supervised steel construction for the Ambassador Bridge in Detroit and the Hell Gate Bridge in New York City.

## CONSTRUCTION STATISTICS FOR THE WEEK

ENGINEERING construction awards for the short week due to Armistice Day total $38,372,000, compared with the previous short week of $30,071,000. This week's total is made up of private, $18,740,000; public, $19,632,000; federal, $3,168,000; state and municipal, $16,464,000. * Corresponding values for a year ago are: Total, $28,773,000; private, $6,479,000; public, $22,294,000; federal, $4,597,000; state and municipal, $17,697,000.

Private awards this week make up 51 per cent of the total compared with an average of 26 per cent in 1932; 30 per cent in 1933; 23 per cent in 1934 and 1935 and an average to date in 1936 of 26 per cent. This increasing proportion of private work is a strengthening trend.

For this week, waterworks awards are higher as are industrial buildings, commercial buildings, earthwork, waterways and drainage, and highways. The classified totals in dollar volume are: Industrial buildings, $5,337,000; commercial buildings, $7,172,000; public buildings, $2,151,000; highways, $8,017,000; bridges, $1,755,000; sewerage, $1,135,000; waterworks, $1,800,000; earthwork, drainage, $2,750,000; unclassified types of work, $8,255,000.

The larger awards for the week include: Improvements and extensions to plant, force account, San Diego Consolidated Gas and Electric Co., San Diego, Calif., $2,200,000; apartments, City and Suburban Homes Co., Long Island City, N. Y., $1,200,000; apartment for Fillmore Gardens, Inc., Elmhurst, N. Y., $500,000; highways, Ohio, $607,000; Mississippi, $1,374,000; improving 5th Street, from Hartman to Riverton Streets, McKeesport, Pa., $739,000; completing University Mound Reservoir, San Francisco, Calif., $693,000; repairing jetty at mouth Umpqua River, U. S. Engineers, Portland, Ore., $939,000; 250,000 bbl. cement for Conchas Dam, Bureau of Reclamation, Tucumcari, N. Mex., $660,000; gasoline driven dumptrucks, New York, N. Y., $1,590,000; 55,162 tons steel rails and other equipment, Southern Pacific Co., Los Angeles, Calif., $3,410,000; 48,750 tons steel rails, Chicago and Northwestern Ry., Chicago, Ill., $1,900,000.

New capital includes state and municipal bond sales, $1,547,000 and PWA allotments, $14,356,000.

### CONTRACTS
(Thousands of Dollars)

| | Weekly Average Nov. 1935 | Prev. 4 Weeks | Week Nov. 12 1936 |
|---|---|---|---|
| Federal Government | $3,797 | $3,433 | $3,168 |
| State and Municipal | 28,871 | 21,538 | 16,464 |
| Total public | $32,668 | $24,971 | $19,632 |
| Total private | 6,075 | 11,231 | 18,740 |
| Week's total | $38,743 | $36,202 | $38,372 |

Cumulative to date:
1935 $1,261,136,000 1936 $2,026,244,000

### NEW PRODUCTIVE CAPITAL
(Thousands of Dollars)

| 1936 | Week Nov. 12 | Cumulative |
|---|---|---|
| State and municipal | $1,547 | $434,892 |
| PWA non-federal | 14,356 | 150,884 |
| RFC loans | | 26,663 |
| Corporate issues | | 210,138 |
| PWA loans, Private | | —1,665* |
| Total Non-Federal | $15,903 | $820,912 |
| Federal | | 691,237 |
| Federal | | |
| Total new capital | $15,903 | $1,512,149 |

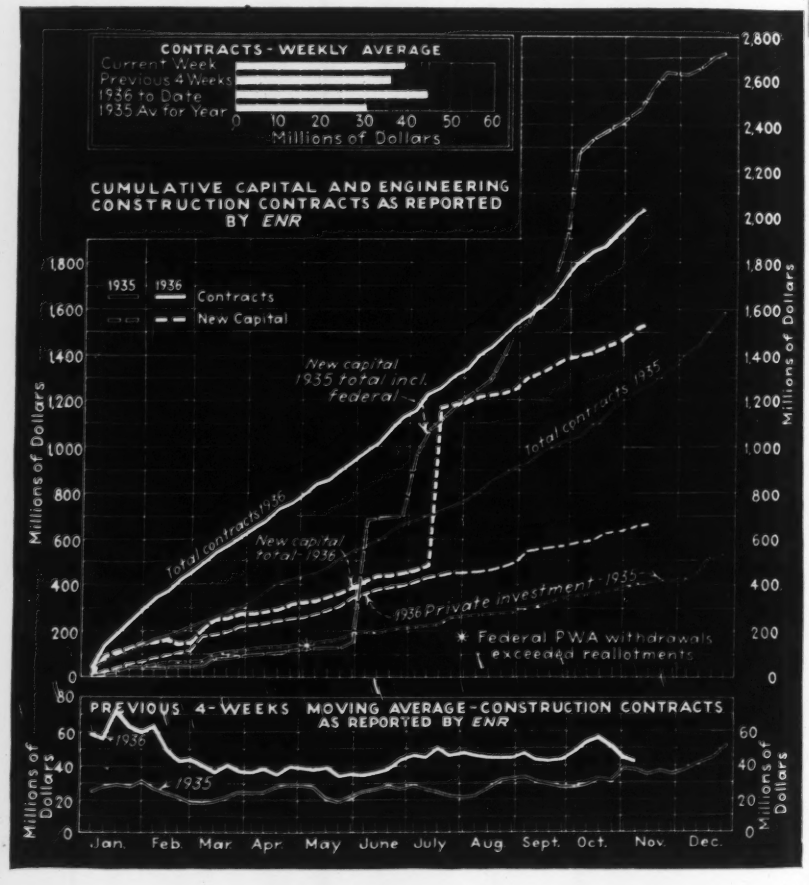
Cumulative to date:
1935 $2,567,442,000 1936 $1,512,149,000

*Bond sales in this classification exceed reallotments during current year.

Note: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities. RFC direct purchase of bonds for "self liquidating" projects, and 25 per cent of WPA construction appropriations.

### INDEX NUMBER

| ENR Cost | 1913 = 100 | 1926 = 100 | ENR Volume | 1913 = 100 | 1926 = 100 |
|---|---|---|---|---|---|
| **Nov., 1936** | **212.70** | **102.24** | **Oct., 1936** | **178** | **78** |
| Oct., 1936 | 211.50 | 101.67 | Sept., 1936 | 200 | 88 |
| Nov., 1935 | 194.90 | 93.69 | Oct., 1935 | 162 | 71 |
| 1935 (Av.) | 195.22 | 93.84 | 1935 (Av.) | 135 | 58 |
| 1934 (Av.) | 198.10 | 95.23 | 1934 (Av.) | 114 | 50 |
| 1933 (Av.) | 170.18 | 81.80 | 1933 (Av.) | 102 | 47 |


CONTRACTS - WEEKLY AVERAGE
Current Week
Previous 4 Weeks
1936 to Date
1935 Av for Year
0 10 20 30 40 50 60
Millions of Dollars
CUMULATIVE CAPITAL AND ENGINEERING CONSTRUCTION CONTRACTS AS REPORTED BY ENR
1935 1936
Contracts
New Capital
New capital 1935 total incl. Federal
Total contracts 1935
Total contracts 1936
New capital total 1936
1936 Private investment 1935
* Federal PWA withdrawals exceeded reallotments
Millions of Dollars
PREVIOUS 4-WEEKS MOVING AVERAGE - CONSTRUCTION CONTRACTS AS REPORTED BY ENR
1936
1935
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

# Current Construction Unit Prices

## Black Warrior River Dam, Birmingham

ALTERNATIVE bids taken for an impounding dam, 1,500 ft. long and 185 ft. high, on the Black Warrior River 40 miles above Birmingham, Ala., reflect a 7.75-per cent saving for a rockfill as compared with a concrete structure. The work is a part of the city's new industrial water system financed through PWA funds, water being brought into the city through a 60-in. steel pipe main awarded under previous contract as noted in *ENR*, Oct. 15, 1936. Seven bids were submitted for the rockfill structure ranging from a low of $1,685,231 to a high of $2,622,298, with four bids on the concrete alternative ranging from $1,826,840 to $2,490,237. Despite the saving effected on the rock dam in the award, three of the same bidders on both alternatives were 5 per cent to 25 per cent higher for the rock dam than for the concrete.

The following tabulation lists the units and totals of the three low bidders: (A) Walsh Const. Co. (contract) Davenport, Iowa, $1,685,231; (B) Martin Wunderlich Co., Jefferson City, Mo., $1,932,687; (C) S. J. Groves & Sons Co., Minneapolis, Minn., $1,998,777.

BLACK WARRIOR RIVER DAM, BIRMINGHAM, ALA.

| Item | A | B | C |
|---|---|---|---|
| 1. Grubbing—5.2 acres | $100.00 | $200.00 | $100.00 |
| 2. Rock excavation—14,500 cu. yd. | 2.50 | 2.00 | 3.00 |
| 3. Earth excavation—243,000 cu. yd. | .40 | .30 | .50 |
| 4. Rock fill—911,000 cu. yd. | .67 | .92 | 1.00 |
| 5. Clay fill—621,440 cu. yd. | .40 | .30 | .28 |
| 6. Overhaul—1,280,000 sta. yd. | .006 | .01 | .01 |
| 7. Riprap—38,300 sq. yd. | .45 | .60 | 1.00 |
| 8. Tunnel and shaft exca.—21,620 cu. yd. | 10.39 | 14.00 | 11.00 |
| 9. Concrete, cl. A—6,921 cu. yd. | 22.50 | 20.00 | 24.00 |
| 10. Concrete, cl. B—19,000 cu. yd. | 13.05 | 15.00 | 12.50 |
| 11. Reinforcing steel—79,300 lb. | .04 | .05 | .05 |
| 12. Structural steel—440,300 lb. | .06 | .06 | .08 |
| 13. Drain pipe, fittings, 4 in.—50 lin. ft. | 1.00 | 2.50 | 1.50 |
| 14. Drain pipe, fittings, 3 in.—1,636 lin. ft. | .69 | 1.30 | 1.00 |
| 15. Drain pipe, fittings, 1½ in.—110 lin. ft. | .40 | 1.00 | .50 |
| 16. Grout pipe, 1½ in.—750 lin. ft. | .50 | 1.00 | .50 |
| 17. Drill grout, weep holes—3,800 lin. ft. | 1.10 | 1.00 | 2.00 |
| 18. Drill grout holes for weep holes—1,100 lin. ft. | .30 | 1.00 | 1.00 |
| 19. Install sluice gates, appurtenances—20 ton | 75.00 | 50.00 | 100.00 |
| 20. Lay steel pipe, specialties, 60 in.—674 lin. ft. | 2.00 | 5.00 | 5.00 |
| 21. Misc., steel—2,200 lb. | .25 | .25 | .15 |

## Boca Dam, Truckee River

CONTRACT for the construction of an earth dam across the Little Truckee River, 20 miles southwest of Reno, Nev., was awarded in October, 1936. The work consists of a 1,650-ft. dam, 110 ft. high at the crest, made up of a moistened and rolled enbankment of clay, sand and gravel, with the downstream slope covered with a heavy rock fill and the upstream slope with 3 ft. of riprap. Appurtenant works include a concrete-lined tunnel under the right abutment, through which the river will be diverted during the dam construction, outlet works, and an open channel spillway at the left abutment. Completion is scheduled within 800 days with minimum PWA wages stipulated of $1.10 per hr. for skilled and 45c. per hr. for common labor. The Bureau of Reclamation furnishes concrete aggregate at the site; and cement, reinforcing steel, fittings for permanent installations, drain and sewer pipe, gate and operating works mechanisms, installed machinery and miscellaneous iron works, f.o.b. cars Boca, Calif.

The units and totals of the three low bids of the eighteen submitted are presented in the following tabulation: (A) Geo. W. Condon Co., Omaha, Neb. (contract) $729,435; (B) D. McDonald and MacDonald & Kahn Co., Ltd., San Francisco, Cal., $740,550; (C) S. J. Groves & Sons Co., Minneapolis, Minn., $749,050.

BOCA DAM, TRUCKEE RIVER STORAGE PROJECT

| Item | A | B | C |
|---|---|---|---|
| 1. Divert river, unwater foundation—total | $7,500.00 | $2,000.00 | $3,500.00 |
| 2. Excavate, strip borrow pits—65,000 cu. yd. | .25 | .14 | .25 |
| 3. Excavate, strip emb.—160,000 cu. yd. | .40 | .43 | .40 |
| 4. Exc., spillway, tunnel inlet, outlet—56,000 cu. yd. | .25 | .26 | .80 |
| 5. Exc., rock, spillway, tunnel, inlet, outlet—19,500 cu. yd. | .90 | .75 | .80 |
| 6. Exc., misc., outlet tunnels—5,000 cu. yd. | 11.00 | 7.00 | 12.00 |
| 7. Exc., emb. drains, cut-off trenches—11,000 cu. yd. | .25 | .40 | .60 |
| 8. Exc., rock, emb. drains, cut-off trenches—300 cu. yd. | 4.00 | 4.00 | 6.00 |
| 9. Exc., cutoff walls, trenches—300 cu. yd. | 3.00 | 3.00 | 3.00 |
| 10. Exc., rock, concrete cutoff ftgs., trenches 600 cu. yd. | 4.00 | 3.00 | 8.00 |
| 11. Exc., common, borrow pits, transport emb.—1,000,000 cu. yd. | .22 | .21 | .19 |
| 12. Exc., rock, borrow pits, transport emb., spillway—110,000 cu. yd. | .80 | .80 | .65 |
| 13. Backfill—5,600 cu. yd. | .80 | 1.00 | .50 |
| 14. Earthfill, emb.—870,000 cu. yd. | .05 | .10 | .07 |
| 15. Rockfill, downstream slope—115,000 cu. yd. | .20 | .20 | .22 |
| 16. Riprap, upstream, slope, spillway channel—31,000 cu. yd. | .40 | .35 | .35 |
| 17. Dumped riprap, spillway channel—1,200 cu. yd. | .80 | .60 | .40 |
| 18. Screened gravel, spillway channel—600 cu. yd. | 2.00 | 3.00 | 2.00 |
| 19. Construct sewer pipe drains, 12 in.—700 lin. ft. | .80 | 1.00 | 1.00 |
| 20. Construct sewer pipe drains, 8 in.—2,300 lin. ft. | .75 | .90 | .75 |
| 21. Construct sewer pipe drains, 6 in.—1,800 lin. ft. | .65 | .80 | .60 |
| 22. Construct sewer pipe drains, 4 in., porous conc.—50 lin. ft. | .40 | 1.50 | 1.00 |
| 23. Drill weep holes—350 lin. ft. | .50 | 3.00 | 1.00 |
| 24. Drill grout holes to 25 ft.—2,700 lin. ft. | 1.00 | 1.30 | 1.00 |
| 25. Drill grout holes, 25 to 50 ft.—1,800 lin. ft. | 1.50 | 2.25 | 2.00 |
| 26. Drill grout holes, 50 to 100 ft.—2,000 lin. ft. | 2.00 | 3.25 | 3.00 |
| 27. Install grout pipe, fittings—1,900 lbs. | .15 | .15 | .05 |
| 28. Pressure grouting—6,200 cu. ft. | .90 | 1.00 | 1.00 |
| 29. Drill holes, anchor bars and grouting bars—1,500 lin. ft. | 1.00 | 1.00 | 1.00 |
| 30. Concrete, cut-off walls, trenches, sloping sides—1,300 cu. yd. | 12.00 | 8.00 | 10.00 |
| 31. Concrete, cut-off walls, trenches, vert. sides—250 cu. yd. | 7.00 | 4.00 | 10.00 |
| 32. Concrete, trash rack struct. and transition—110 cu. yd. | 15.00 | 25.00 | 16.00 |
| 33. Concrete, tunnel lining—1,400 cu. yd. | 16.00 | 14.00 | 19.50 |
| 34. Concrete, gate chamber—400 cu. yd. | 20.00 | 18.00 | 19.50 |
| 35. Concrete, spillway floors, outlet work channels—1,680 cu. yd. | 7.00 | 4.00 | 8.00 |
| 36. Concrete, except floors, spillway, outlet works—2,330 cu. yd. | 13.00 | 14.00 | 12.00 |
| 37. Concrete, substructure, needle valve house—200 cu. yd. | 14.00 | 10.00 | 20.00 |
| 38. Concrete, superstructure, needle valve house—55 cu. yd. | 14.00 | 25.00 | 30.00 |
| 39. Concrete, parapet and curb walls—630 cu. yd. | 9.00 | 15.00 | 15.00 |
| 40. Place reinforcing bars—670,000 lb. | .017 | .02 | .015 |
| 41. Install Metal sealing strips—600 lin. ft. | .50 | .55 | .50 |
| 42. Furnish, install steel tunnel—liner plates—16,000 lb. | .08 | .15 | .10 |
| 43. Special finish, concrete surfaces—2,000 sq. yd. | .50 | 1.00 | .50 |
| 44. Construct needle valve house, except concrete—total | 800.00 | 500.00 | 1,500.00 |
| 45. Install trash rack metal work 28,000 lb. | .02 | .03 | .05 |
| 46. H. P. hydraulic slide gates, metal conduit linings—95,000 lb. | .04 | .04 | .05 |
| 47. Control apparatus h. p. gates—8,000 lb. | .15 | .11 | .10 |
| 48. Install outlet pipes—150,000 lb. | .04 | .05 | .03 |
| 49. Install needle valves—48,000 lb. | .05 | .055 | .05 |
| 50. Install radial gates and operators 40,000 lb. | .04 | .05 | .05 |
| 51. Install metal stairway, walkways, gratings—16,000 lb. | .03 | .11 | .10 |
| 52. Install pipe handrails—4,500 lb. | .03 | .06 | .10 |
| 53. Install misc. metal work—9,000 lb. | .08 | .11 | .10 |
| 54. Install elec. metal conduit, to 1½ in. dia.—3,300 lin. ft. | .25 | .30 | .30 |
| 55. Install elec. conductors, ground wires, apparatus—total | 730.00 | 500.00 | 1,000.00 |

## Little Rock Waterworks Dam

COMPLETION of the new waterworks system of Little Rock, Ark., progresses another step with award of contract in September, 1936, by Burns & McDonnell, supervising engineers, for the construction of an earth dam and spillway. Nine bids were submitted ranging from the contract low of $921,706 to a high of $1,690,139. The following tabulation lists the units and totals of the three low bidders: (A) L. C. Brayton & Co., Dyersburg, Tenn. (contract) $921,706; (B) Martin Wunderlich, Jefferson City, Mo., $990,917; (C) Russ Mitchell, Inc., Houston, Texas, $1,003,647.

| Item | A | B | C |
|---|---|---|---|
| 1. Care of stream—total | $5,000.00 | $5,000.00 | $1,500.00 |
| 2. Clear, grub dam site—total | 7,500.00 | 6,000.00 | 4,000.00 |
| 3. Clear, grub, borrow areas—150 acres | 110.00 | 100.00 | 205.00 |
| 4. Exca. upstream div. channel—22,500 cu. yd. | .20 | .35 | .23 |
| 5. Exca. upstream div. channel, special—10,500 cu. yd. | 1.00 | .40 | .61 |
| 6. Exca. div. conduit, downstream div. channel—17,000 cu. yd. | .20 | .35 | .32 |
| 7. Exca. dam fdn., div. conduit, channel—11,150 cu. yd. | 1.25 | .70 | 1.15 |
| 8. Spillway exca.—23,550 cu. yd. | .30 | .50 | .55 |
| 9. Spillway exca., classified—81,600 cu. yd. | .80 | .80 | .90 |
| 10. Exca. dam fdt., unclass.—182,850 cu. yd. | .20 | .30 | .29 |
| 11. Exca. outlet tower, pipe line connections—65 cu. yd. | 1.00 | 1.00 | 1.00 |
| 12. Exca. outlet tower, pipe line connections, classified—45 cu. yd. | 5.00 | 2.00 | 1.75 |
| 13. Compacted embankment—126,750 cu. yd. | .20 | .295 | .16 |
| 14. Compacted emb., from borrow pits—1,403,500 cu. yd. | .26 | .295 | .33 |
| 15. Overhaul for emb. materials—282,000 sta. yd. | .02 | .02 | .01 |
| 16. Gutters, drains, structures, downstream slope—total | 6,000.00 | 5,000.00 | 6,220.00 |
| 17. Dress with top soil, sodding—total | 5,000.00 | 5,000.00 | 10,300.00 |
| 18. Riprap, hand placed—22,800 sq. yd. | 2.00 | 2.00 | .45 |
| 19. Riprap, dumped, div. channels—710 cu. yd. | .50 | 1.00 | 1.00 |
| 20. Riprap dumped, spillway—26,250 cu. yd. | .40 | .50 | .58 |
| 21. Concrete cl. A, div. conduit, structures—1,920 cu. yd. | 22.00 | 21.00 | 17.70 |
| 22. Reinforced steel—231,090 lb. | .045 | .04 | .045 |
| 23. Pipe valves, m. h. covers, div. conduit—total | 5,000.00 | 7,000.00 | 6,000.00 |
| 24. Concrete, cl. A, spillway, paving, walls, fdns.—8,215 cu. yd. | 15.50 | 15.00 | 14.45 |
| 25. Reinf. steel, spillway structures—488,600 lb. | .045 | .04 | .045 |
| 26. Misc. items, spillway structures—total | 5,000.00 | 11,000.00 | 1,830.00 |
| 27. Concrete, cl. A, intake tower, fdn., pipe line connections—490 cu. yd. | 30.00 | 28.00 | 28.00 |
| 28. Reinf. steel, outlet tower, structures—44,400 lb. | .05 | .05 | .05 |
| 29. Misc. items, gates, valves, screens, etc., outlet tower—total | 11,000.00 | 10,000.00 | 15,250 |
| 30. Bridge across spillway—total | 5,500.00 | 6,000.00 | 5,000.00 |
| 31. Parking plaza, driveway, walks, sodding—total | 8,300.00 | 10,000.00 | 5,625.00 |
| 32. Operator's cottage—total | 11,000.00 | 10,000.00 | 12,250.00 |
| 33. Garage, water wheel struct.—total | 5,200.00 | 5,000.00 | 5,625.00 |
| 34. Elec. power and lighting equip.—total | 5,000.00 | 4,000.00 | 5,250.00 |
| 35. Well and water supply equip.—total | 1,000.00 | 750.00 | 800.00 |
| 36. Fish rearing pond, W. S. pipe line—total | 5,450.00 | 6,000.00 | 6,375.00 |
| 37. Laboratory tests—total | 2,500 | 2,500.00 | 2,500.00 |
| 38. Misc. items—total | 4,000.00 | 100.00 | 1,500.00 |

## Paving, Rhode Island Highways

FOUR contracts awarded by the Rhode Island department of public works during September and October, 1936, for 2½ miles of sheet asphalt and asphaltic concrete paving indicate only slight unit price variations in these types of road construction. The location, length and type of paving, with totals and the unit prices of the three low bidders on each of the four projects are given in the following tabulations.

**I—Central Falls: 0.303 mi. sheet asphalt on concrete base.**
(A) J. J. McHale & Sons, Inc., Pawtucket, R. I. (contract)...$21,649
(B) M. A. Gammino Const. Co., Providence, R. I. ... 23,864
(C) C. W. Blakeslee & Sons, Inc., New Haven, Conn. ... 27,428

**II—Warren: 0.653 mi. asphaltic concrete on cement concrete base.**
(D) Callan Const. Co., Bristol, R. I. (contract)...$39,936
(E) P. Romano & Sons, Warren, R. I. ... 41,633
(F) M. A. Gammino Const. Co., Providence, R. I. ... 42,063

**III—East Providence: 0.65 mi. sheet asphalt on concrete base.**
(G) M. A. Gammino Const. Co., Providence, R. I. (contract)...$59,367
(H) P. Romano & Sons, Warren, R. I. ... 62,463
(J) J. J. McHale & Sons, Inc., Pawtucket, R. I. ... 62,499

**IV—West Warwick: 0.752 mi. asphaltic concrete on cement concrete base.**
(K) M. A. Gammino Const. Co., Providence, R. I. (contract)...$53,735
(L) A. Susi & Co., Rosindale, Mass. ... 56,822
(M) Lane Const. Co., Meriden, Mass. ... 57,110

**I—Central Falls**

| | A | B | C |
|---|---|---|---|
| 1. Earth excavation—2,685 cu.yd. | $0.40 | $0.40 | $0.55 |
| 2. Trench excavation, 5 to 10 ft.—3 cu.yd. | 1.50 | 2.25 | 3.00 |
| 3. Trench excavation, to 5 ft.—27 cu.yd. | 1.00 | 1.50 | 2.00 |
| 4. Remove conc. pavement—128 cu.yd. | 2.00 | 1.00 | 1.00 |
| 5. Trimming, fine grading—7,475 sq.yd. | .05 | .05 | .06 |
| 6. Reinf. conc. pipe, 12 in.—58 lin.ft. | 1.25 | 1.10 | 1.75 |
| 7. Catch basins—7 each | 80.00 | 110.00 | 110.00 |
| 8. Cement conc. base—7,066 sq.yd. | 1.05 | 1.28 | 1.45 |
| 9. Sheet asphalt pavement—7,066 sq.yd. | 1.05 | 1.10 | 1.25 |
| 10. Granite curb—262 lin.ft. | 1.60 | 1.85 | 1.75 |
| 11. Circ. granite curb, 8 ft.—2 each | 30.00 | 28.00 | 27.00 |
| 12. Granite corners, 2 ft.—5 each | 8.00 | 6.50 | 7.00 |
| 13. Remove, reset granite curb—2,702 lin.ft. | .30 | .25 | .35 |
| 14. Granolithic sidewalks—1,718 sq.yd. | 1.60 | 1.75 | 2.00 |
| 15. Circ. granite curb, 30 ft. rad.—70 lin.ft. | 2.00 | 2.25 | 2.25 |
| 16. Reset granite corners—25 each | 2.00 | 2.50 | 4.00 |
| 17. Reset circ. returns—14 each | 6.00 | 5.00 | 8.00 |
| 18. Remove, reset manholes, etc.—9 each | 10.00 | 5.00 | 7.00 |

**II—Warren**

| | D | E | F |
|---|---|---|---|
| 1. Remove trees—7 each | $30.00 | $20.00 | $25.00 |
| 2. Earth excavation—3,384 cu.yd. | .40 | .35 | .40 |
| 3. Trench excavation, to 5 ft.—985 cu.yd. | .75 | .75 | 1.00 |
| 4. Trim, fine grading—10,605 sq.yd. | .04 | .03 | .05 |
| 5. Gravel foundation—500 cu.yd. | .70 | .75 | .60 |
| 6. Reinf. conc. pipe, 12 in.—720 lin.ft. | 1.15 | 1.20 | 1.10 |
| 7. Reinf. conc. pipe, 15 in.—320 lin. ft. | 1.40 | 1.45 | 1.35 |
| 8. Reinf. conc. pipe, 18 in.—428 lin.ft. | 2.00 | 1.90 | 1.80 |
| 9. Reinf. conc. pipe, 24 in.—132 lin.ft. | 3.00 | 2.50 | 2.70 |
| 10. Vitrif. pipe, 12 in.—510 lin.ft. | .75 | .75 | .70 |
| 11. Vitrif. pipe, 15 in.—500 lin.ft. | 1.35 | 1.00 | 1.10 |
| 12. Vitrif. pipe, 18 in.—350 lin.ft. | 1.35 | 1.50 | 1.40 |
| 13. Catch basins, D—12 each | 65.00 | 75.00 | 110.00 |
| 14. Catch basins, E—4 each | 80.00 | 100.00 | 125.00 |
| 15. Drop inlets—2, each | 40.00 | 50.00 | 50.00 |
| 16. Stone masonry—10 cu.yd. | 10.00 | 10.00 | 15.00 |
| 17. Guard rail—305 lin.ft. | .45 | .45 | .40 |
| 18. Cmt. conc. base—9,879 sq.yd. | 1.05 | 1.20 | 1.27 |
| 19. Std. conc. curb—5,418 lin.ft. | .60 | .65 | .70 |
| 20. Remove, reset granite curb—1,243 lin.ft. | .20 | .40 | .25 |
| 21. Granolithic sidewalks—849 sq.yd. | 1.60 | 1.50 | 2.00 |
| 22. Bituminous sidewalks—343 sq.yd. | 1.15 | 1.20 | 1.25 |
| 23. Reset granite corners, 2 ft.—10 each | 1.00 | 1.00 | 2.00 |
| 24. Reinf. conc. driveways, 8 in.—22 cu.yd. | 12.00 | 10.00 | 12.00 |
| 25. Asphalt conc. pavement—9,879 sq.yd. | 1.20 | 1.20 | 1.00 |
| 26. Reset manhole frames, etc.—18 each | 10.00 | 6.50 | 5.00 |

**III—East Providence**

| | G | H | J |
|---|---|---|---|
| 1. Remove trees—8, each | $20.00 | $20.00 | $15.00 |
| 2. Earth excavation—4,787 cu.yd. | .35 | .40 | .40 |
| 3. Trench excavation, to 5 ft.—3,490 cu.yd. | .90 | .90 | 1.10 |
| 4. Trench excavation, 5 to 10 ft.—1,533 cu.yd. | 1.35 | 1.35 | 1.65 |
| 5. Trench excavation, 10 to 15 ft.—5 cu.yd. | 1.80 | 1.80 | 2.20 |
| 6. Trim, fine grading—14,870 sq.yd. | .02 | .04 | .05 |
| 7. Gravel foundation — 100 cu.yd. | .80 | 1.00 | 1.25 |
| 8. Reinf. conc. pipe, 12 in.—1,321 lin.ft. | 1.10 | 1.15 | 1.10 |
| 9. Reinf. conc. pipe, 36 in.—30 lin.ft. | 4.35 | 6.00 | 7.00 |
| 10. Reinf. conc. pipe, B, 12 in.—300 lin.ft. | .75 | .80 | .85 |
| 11. Reinf. conc. pipe, B, 18 in.—1,923 lin.ft. | 1.35 | 1.40 | 1.50 |
| 12. Reinf. conc. pipe, B, 20 in.—215 lin.ft. | 1.75 | 1.85 | 2.00 |
| 13. Reinf. conc. pipe, B, 24 in.—681 lin.ft. | 2.35 | 2.25 | 2.50 |
| 14. Reinf. conc. pipe, B, 30 in.—286 lin.ft. | 3.25 | 3.90 | 4.00 |
| 15. Reinf. conc. pipe, B, 36 in.—220 lin.ft. | 4.00 | 5.00 | 5.00 |
| 16. Catch basins, D—29, each | 80.00 | 75.00 | 100.00 |
| 17. Catch basin chute—1, each | 100.00 | 100.00 | 115.00 |
| 18. Manholes—13, each | 100.00 | 100.00 | 80.00 |
| 19. Manholes, frames, covers—15, each | 25.00 | 35.00 | 32.00 |
| 20. Drop inlet—1, each | 40.00 | 50.00 | 50.00 |
| 21. Concrete masonry—7 cu.yd. | 20.00 | 18.00 | 20.00 |
| 22. Concrete base course, 7 in.—14,870 sq.yd. | 1.00 | 1.15 | 1.05 |
| 23. Sheet asphalt binder—14,870 sq.yd. | 1.10 | 1.10 | 1.05 |
| 24. Granite curb—100 lin.ft. | 1.30 | 1.75 | 2.00 |
| 25. Granite corners, 2 ft.—6, each | 7.00 | 7.50 | 7.00 |
| 26. Remove, reset granite curb—6,164 lin.ft. | .25 | 1.25 | .30 |
| 27. Granolithic conc. sidewalk—726 sq.yd. | 1.60 | 1.75 | 1.70 |
| 28. Bituminous conc. sidewalk—2,090 sq.yd. | 1.20 | 1.15 | 1.20 |
| 29. Remove present conc. blocks—2,264 sq.yd. | .30 | .40 | .30 |
| 30. Granite inlets, B—6, each | 70.00 | 60.00 | 50.00 |
| 31. Granite inlets, C—8, each | 70.00 | 65.00 | 50.00 |
| 32. Terminal catch basin—1, each | 550.00 | 125.00 | 125.00 |
| 33. Special conc. edging—320 lin. ft. | .50 | .40 | .50 |

**IV—West Warwick**

| | K | L | M |
|---|---|---|---|
| 1. Earth excavation—7,926 cu.yd. | .40 | .40 | .50 |
| 2. Trench excavation—408 cu.yd. | 1.25 | 1.00 | 1.50 |
| 3. Trim, fine grading—14,949 sq.yd. | .03 | .05 | .05 |
| 4. Gravel foundation—2,423 cu.yd. | .60 | .75 | .50 |
| 5. Culvert pipe, A, 12 in.—750 lin.ft. | 1.10 | 1.25 | 1.25 |
| 6. Culvert pipe, A, 18 in.—210 lin.ft. | 1.75 | 2.00 | 2.00 |
| 7. Catch basins, D—11, each | 90.00 | 75.00 | 90.00 |
| 8. Cmt. conc. base, 7 in.—14,815 sq.yd. | 1.15 | 1.25 | 1.30 |
| 9. Asphalt conc. binder—14,815 sq.yd. | 1.20 | 1.10 | 1.30 |
| 10. Std. conc. curb—7,796 lin.ft. | .70 | .90 | .75 |
| 11. Granolithic conc. sidewalk—1,416 sq.yd. | 1.75 | 2.40 | 1.60 |
| 12. Revamp catch basin tops, etc.—22, each | 40.00 | 25.00 | 20.00 |
| 13. Curb inlets, conc. B—5, each | 70.00 | 60.00 | 45.00 |
| 14. Curb inlets, conc. C—5, each | 70.00 | 50.00 | 45.00 |
| 15. Catch basins, remove, backfill—1, each | 10.00 | 20.00 | 10.00 |
| 16. Remove, reset manhole cover frames—12, each | 10.00 | 25.00 | 20.00 |
| 17. Revamp inlet tops—6, each | 25.00 | 20.00 | 20.00 |
| 18. Remove drop inlets, backfill—3, each | 10.00 | 20.00 | 20.00 |
| 19. Remove, haul granite curb—7,346 lin.ft. | .20 | .25 | .05 |